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REPORT ON STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS

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REPORT ON STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
CONTRACT NUMBER 12-10-001-778

Mueser, Rutledge, Wentworth & Johnston
Consulting Engineers
415 Madison Avenue, New York, N. Y.

June 1968

FORWORD

Observations of movements of articulated concrete outlet pipe beneath twenty earth dams of the Soil Conservation Service were taken by that organization and were made available to Mueser, Rutledge, Wentworth and Johnston for analysis. This study was performed under the direction of P. C. Rutledge, Partner, and J. P. Gould, Associate. The finite element analysis of movements beneath earth dams was derived by W. J. Thompson and the computer solution was programmed by him. This theoretical study was materially assisted by the earlier work of R. J. Woodward, III, referenced in the text. Coordination of all elements of the study was performed by J. P. Gould and he also prepared the basic text of this report.

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CONTENTS

STUDY OF MOVEMENTS OF ARTICULATED CONDUITS BENEATH EARTH DAMS ON COMPRESSIBLE FOUNDATIONS

1. INTRODUCTION

1.1 Authorization

This report is submitted pursuant to Contract No. 12-20-001-778, dated June 6, 1967, between the Soil Conservation Service (SCS), U. S. Department of Agriculture, and Mueser, Rutledge, Wentworth & Johnston, Consulting Engineers. That Contract for engineering services provides for a program to analyze and evaluate observations of horizontal and vertical movement made on concrete pipe conduits in dams of the Soil Conservation Service. This report summarizes the studies made under the Contract.

1.2 Scope of Work

The Contract divides the work in the following six phases:

Phase I, Evaluation of Data. Study and evaluate data from 20 earth dams and develop a standard form for presentation of data.

Phase II, Analysis of Data and Establishment of Relationships. Analyze the data from Phase I and establish relationships between foundation consolidation and relative movement and rotation of the ends of sectional pipe by methods similar to those presented in the report of Moran, Proctor, Mueser and Rutledge for the American Concrete Pressure Pipe Association, entitled, "Report on Investigation of Deformations in Foundations of Earth Embankments Containing Concrete Pressure Pipe Conduits".

Phase III, Evaluation of Interim Report. Evaluate the analyses and conclusions contained in the report by H. L. Cappleman Jr., entitled "Interim Report of Field Study of Movements of Inlet and Barrel Sections of Principal Spillways Under Earth Dams on Compressible Foundations".

Phase IV, Recommendations for Design Criteria Procedures. Prepare recommendations for the calculation of conservative requirements for joint extensibility and rotation capacity of sectional conduits under earth dams.

Phase V, Development of Computer Program. Develop a computer program applicable to the type of analysis in Phase II and determine what correlations with field observations can be obtained.

Phase VI, Report. Prepare a final report covering Phases I through V and including recommendations for continued use of the data and the procedures developed.

1.3 Statement of the Problem

It has long been recognized that the loading created by an earth dam on a soil foundation will produce measureable horizontal and vertical displacement within the embankment and its foundation. In recent years, with the development of devices such as the "slope indicator", the available field observations indicate complex patterns of horizontal displacements which depend on a combination of conditions, such as, properties of the soils involved, configuration of the foundation in the stream valley, sequence of construction and the relation between applied stresses and soil strength. In SCS flood control dams these displacements are manifested by vertical and horizontal movements of jointed concrete pipe placed as low-level outlet works. Excessive movements of articulated conduits could result in disjoining the pipes or cracking the conduit which might impair the functioning of the outlet or even endanger stability of the dam since the conduits are intended to flow full under pressure in some circumstances. As a consequence, SCS has taken readings of movements in conduit pipes beneath selected structures over a period of about ten years, and since 1961, has systematized collection of data for certain dams scattered throughout their nationwide projects. The purpose of the observation program is to permit the development of procedures to estimate joint movement with reasonable accuracy during design. While the information was obtained primarily from low to moderate height dams, it represents a far more detailed and reliable collection of observations than is ordinarily available for analysis of soil mechanics problems and is well worth an intensive evaluation effort.

1.4 Previous Studies

References to the literature describing the theoretical background of the problem, observations by others, and analytical techniques are listed following the main body of the text. The specific area of this study is part of the larger problem of deformations within and beneath earth structures which is receiving increased attention as methods for field observation of movement are refined. Section A of the References concerns data on the extension of low-level conduits. Section B provides examples of field observations of horizontal movements in earth masses.

An analysis of the conduit extension problem was presented in the Moran, Proctor, Mueser & Rutledge report of September 1960 concerning deformations in foundations of earth embankments, which is listed as Reference 2 and is referred to herein as "the report of September 1960". This work was based on earlier theoretical studies listed in Section C of the References. The tentative method contained in that report for evaluating horizontal movements has been utilized by SCS as a routine procedure for predicting the performance of conduits beneath earth embankments.

Following the initiation of standardized field observations on SCS conduits in 1961, an interim report was completed in May 1965 by H. L. Capplemen Jr. entitled "Field Study of Movements of Inlet and Barrel Sections of Principal Spillways Under Earth Dams on Compressible Foundations", Reference 4. This study was later summarized by the same author in Paper No. 5548 of the Journal of the Soil Mechanics and Foundations Division, ASCE, "Movements in Pipe Conduits Under Earth Dams", Reference 5. Cappleman's conclusions emphasized the importance of the relation between applied stress and foundation strength in evaluating conduit extension.

An instructive example of the significance of deformations in earth dams was presented by Linnel and Shea in Reference 6. In this case the considerable horizontal movement on the upstream slope of Otter Brook Dam led to a detailed study of properties of the glacial till which had been utilized in the embankment. This was followed by an analysis by Clough and Woodward utilizing the recently developed finite element technique which explained the magnitude of the observed movements, Reference 13.

Since 1960 an effort of great potential value has been put forward in utilizing computer solutions for stresses and deformations in earth masses derived by the finite element method. This application has been developed intensively at the University of California and the studies in Sections D and E of the Reference describe the procedure and give examples of its application. The doctorial dissertation of R. J. Woodward, III, "Analysis of Stresses and Displacements in Embankments with Non-Linear Material Properties," Reference 14, was provided to us by the author and has been particularly useful in this study.

1.5 Organization of This Report

Theoretical aspects of the conduit extension problem are presented in Chapters 2 and 3. Chapter 2 reviews conclusions from previous work while Chapter 3 summarizes the results obtained from the finite element analysis developed under this contract and compares them with the previous work. The derivation of the finite element solution and its application to a computer program are described in Appendix A which is bound following the text and tables. Complete program cards and the print-out sheets for two computer solutions were submitted separately to SCS on March 8, 1968. The field observation program initiated in 1961 by SCS is described in Chapter 4. These observations made on conduits in twenty earth dams are plotted in summary form on the Drawings Nos. 1 through 20 which appear after Appendix A and plates. Brief descriptions of the twenty case histories are given in Chapter 5. The observation data, the interpreted data and the embankment and foundation conditions are listed in Tables Nos. 4, 5 and 6. The relationship of the observed movements to the theoretical factors are examined in Chapter 6 and conclusions from the Cappleman report are considered at this point. Recommendations for methods of predicting conduit movement in advance of construction are given in Chapter 7.

2. -THEORETICAL CONCEPTS OF EMBANKMENT DEFORMATIONS

2.1 General Relationships

The MPM&R September 1960 report summarized information then available relating to embankment and foundation deformation, largely from References 9 through 12. A tentative method for estimating conduit extension was derived from work of the Prairie Farms Rehabilitation Administration in Reference 12. The principal conclusions developed in the September 1960 report are summarized in the following paragraphs. The definitions and symbols utilized in the September 1960 report and in this study relating to embankment and foundation geometry, material properties and components of movement are listed in Table No. 1. These basic factors are illustrated on Plate No. 1.

Stability of a triangular-shaped earth or rock fill dam requires development of shear stresses on the embankment-foundation interface which restrain the tendency of the embankment to spread due to outward directed horizontal pressures originating in the central portion of the dam. If vertical stresses alone were applied to the interface, the foundation would bend as a slab and at intermediate values of Poisson's ratio between the limits of zero and 0.5 and there would be little tendency for either extension or compression in a horizontal direction on the interface. The observed lengthening of conduits under observation could not be explained by the increase in length caused solely by settlement of the conduit in the form of a catenary. However, vertical pressures plus shear stresses applied at the upper surface of a compressible foundation will produce a profile of vertical settlement on the interface as well as horizontal displacements directed outward from the axis of the dam towards the toes of the slopes.

The most promising elastic analysis available for the September 1960 report, Reference 12, considers the application of vertical pressures plus two symmetrical triangular shear diagrams at the foundation interface. The effect of this combined loading is to produce horizontal tensile strains along the interface in the central portion of the embankment and compressive strains beyond the mid-point of the embankment side slope. This solution was converted to values of maximum unit horizontal strain for intermediate values of Poisson's ratio on Plate No. 12 of the September 1960 report. This maximum unit horizontal strain is directly proportional to the magnitude of the maximum vertical load of the embankment and inversely proportional to the foundation modulus of elasticity. It decreases as the ratio of the dam base width to height increases and it decreases as the ratio of dam base width to depth of compressible foundation increases. It increases as Poisson's ratio increases.

Vertical strain beneath the dam axis due to compression of the foundation soils is directly proportional to the maximum applied pressure, inversely proportional to the foundation elastic modulus and decreases with an increase of Poisson's ratio. It was concluded that, while it was not now practical to determine an accurate elastic modulus for the foundation soils from production testing, it was reasonable to expect that an accurate settlement analysis would yield a reliable value of average vertical strain. Thus, if a working relationship between maximum horizontal and vertical strain could be established, the maximum horizontal strain could be estimated from the results of a conventional settlement analysis. The average vertical strain under triangular vertical pressures applied to the elastic foundation was divided into the theoretical maximum horizontal strain for an intermediate value of Poisson's ratio, yielding the family of curves in Plate No. 29 of the September 1960 report. This strain ratio was plotted as a function of the parameters b/d, b/h and E/p. This relationship was suggested as a tentative method for predicting conduit extension during design studies. A central portion of the diagram, covering the spread of parameters applicable to low to moderate height earth dams, is reproduced as Plate No. 2. Typical values of the strain ratio lie in the range between 0.1 and 0.4.

2.2 Assumptions for Elastic Analysis

The elastic analysis of the September 1960 report and the finite element procedure derived for this study are applied to the conduit extension problem utilizing the following assumptions:

1. The conduit undergoes the same displacements as the surrounding earth materials on the embankment-foundation interface.

2. Both foundation and embankment soils are linear, elastic, isotropic and homogeneous. The September 1960 analysis takes no account of the character of the embankment material except that its unit weight is a factor in the embankment loading diagram. In the finite element analysis the embankment is also considered to be linear elastic but with modulus of elasticity not necessarily equal to that of the foundation.

3. The embankment-foundation cross section is everywhere the same and the same cross section extends a great distance in a direction at right angles to the vertical plane of the conduit. Deformations occur as plane strain throughout and no movement takes place at right angles to the vertical plane through the conduit.

4. The embankment-foundation interface is horizontal and the rigid lower boundary of the foundation is horizontal.

5. Water forces in the reservoir are ignored and the activating force is derived from the weight of the embankment only.

6. The ratio of applied shear stresses to foundation shear strength is sufficiently low that a plastic state of stress does not develop within the foundation.

The conditions required for Item 6 are approximately met if shear strength exceeds shear stress at all points. Jurgenson has shown, Reference 11, that for application of a vertical embankment load only, the shear strength will everywhere exceed shear stress if the maximum vertical pressure, p is less than:

$\left(\frac{s}{2}\right)\left(\frac{b}{d}\right)$ where s = foundation shear strength

If consideration is given to the outward directed shear stresses applied by the embankment loading, p must be less than the following approximate value:

$$\left(\frac{s}{2}\right)\left(\frac{b}{d}\right) - \frac{\gamma_f h^2}{4d}$$

If plastic conditions are developing, the tension zone on the interface tends to expand so that tensile strains occur over an increasing portion of the base width of the embankment.

2.3 Comparison With Field Observations

For the September 1960 report, the records of observations on conduits in nine SCS dams were made available. In these nine cases a survey was made several years after the end of construction of conduit settlement and joint openings. However, no observations were available to determine the initial as-placed condition of the pipe. In this circumstance the settlement had to be taken as the difference in elevation from planned grade and the joint movement was taken equal to the total joint opening. Heights of the embankments ranged from 36 to 57 feet, observed settlements from 0.3 to 3 feet. The median value of maximum horizontal strain was 0.01, and the vertical strain 0.05, equivalent to a maximum strain ratio of 0.2. The theoretical strain ratio for the median embankment dimensions equaled 0.2, assuming a Poisson's ratio of 0.25 which provided the best fit between observed and theoretical values. However, theoretical values were distinctly lower than the observed strain ratio in those cases with the largest joint openings. The diagram for predicting the maximum horizontal strain, reproduced in Plate No. 2, was described as tentative only, to be verified by more complete field observations. It was noted that meaningful observations must include initial measurement of conduit conditions as placed so that later readings could be converted to net values of movement.

3.-ANALYSIS BY THE FINITE ELEMENT METHOD

3.1 Derivation of the Method

A theoretical solution of the conduit extension problem was devised for this study utilizing the finite element method. The purpose was to determine the interface movements without making arbitrary assumptions for embankment loads applied to the foundation and to consider the effects of differences between the embankment and foundation elastic modulus. The general assumptions are those for elastic conditions of plane strain as stated in Section 2.2. The derivation of the finite element analysis and its application to a computer solution are described in Appendix A and illustrated in two plates, Nos. A1 and A2.

The finite element derivation was based largely on the example contained in References 13 and 14. The intent was to provide a flexible program which included consideration of factors in the embankment geometry, such as crest width and berms on the slopes. However, a large amount of the available computer and personnel time was expended in eliminating program bugs. As a result the computer work was limited to a total of twelve cases for a simplified embankment which were intended to investigate the influence on interface movements of the most important parameters. It should be noted, however, that the computer program developed can accommodate the more realistic embankment geometry shown by the upper section on Plate No. A2 as well as variations in embankment slopes.

3.2 Results of the Analysis

The cases studied are grouped in four sets, numbered I through IV, and the basic conditions are listed in Table No. 2. In all cases the embankment was approximated by a simple symmetrical triangle with side slopes of three horizontal to one vertical. In all cases except No. IV the embankment height at the crest was taken as 40 feet. The modulus of elasticity of the embankment material was assumed to equal 100 kips per square feet. Poisson's ratio was taken equal in the embankment and foundation. For convenience, the embankment total unit weight was taken at 100 pounds per cubic foot.

The first group, Nos. I-1 through I-5, was intended to investigate the effect of changing Poisson's ratio, holding the embankment height and foundation depth equal and the modulus of elasticity constant throughout. The median condition used as a standard in all groups is Case No. I-3 with Poisson's ratio of 0.35. The second group, Nos. II-1 through II-4 and including I-3, was intended to investigate the effect of varying the ratio of foundation to embankment modulus of elasticity, holding embankment height and foundation depth equal and Poisson's ratio constant at 0.35. The third group, Nos. III-1 and III-2 and including I-3, concerns the effect of varying the foundation depth between 20, 40 and 60 feet while holding the embankment height constant at 40 feet. The last case, No. IV, was taken to investigate the scale effect on displacements by assuming a similar embankment with height and foundation depth equal to 60 feet.

Displacements obtained from computer solutions of the 12 cases are summarized in Plates Nos. 3 through 7. Cases are grouped to show the effect of varying the several parameters. Displacements are plotted for the interface and for a horizontal line in the foundation at a depth of 10 feet below the interface. Settlements are shown only along the interface. Values are plotted for one-half of the symmetrical embankment to a distance of 90 feet or 3/8 (b) beyond the toe of the embankment. The position of the axis of the dam is at the right margin of the graph. The computer values obtained on each side of the axis of symmetry were averaged to minimize the inconsistencies which remained even after debugging the program. Maximum settlement and maximum horizontal displacement at the interface for the computer cases are summarized in Plates Nos. 8 to 10, plotted against the independent variable being considered for each group. Maximum horizontal strain was determined from the slope of the displacement curve at the axis. Maximum average vertical strain was obtained by dividing maximum settlement by depth, d. The ratio of maximum horizontal to vertical strain is plotted in the upper panel of the Plates Nos. 8 to 10.

3.3 Information From the Analysis

The absolute values of settlement and displacement derived from the finite element analysis are not intended to be generally applicable since they relate to a specific simplified structure. However, these results combined with data referenced in Chapter 2 define the effects of the principal independent variables on the components of movement of the interface.

The significant parameters comprise both geometric factors and material properties, as follows:

Geometry

1. Overall scale factor, F;
2. Height of embankment, h;
3. Base width of embankment, b;
4. Depth of compressible foundation, d.

Material properties

5. Embankment unit weight, γ_f ;
6. Elastic modulus for embankment and foundation, E_e and E_f;
7. Poisson's ratio of embankment and foundation, ν.

The influence of details of the embankment cross-section, such as crest width, broken slopes and berms have not been studied, although these could be included in the basic computer program. It should be recalled that this analysis applies to a range of essentially elastic performances as restricted in Section 2.2.

The six factors of displacement at the interface which were studied are as follows:

1. Maximum settlement, δ ;
2. Maximum vertical strain, $\delta/d = \epsilon_v$;
3. Maximum horizontal displacement, L_h;
4. Maximum horizontal strain, ϵ_h;
5. Ratio of maximum displacements, L_h/δ ;
6. Ratio of maximum strains, ϵ_h/ϵ_v.

The influence of the seven independent variables on the six displacement factors are summarized in Table No. 3 and are discussed in the following subsections.

3.31 Influence of Scale Factor, F. The scale factor equals the ratio of values of h or b or d for two dams and foundations of different size but exactly similar shapes. The effect of a change of scale is illustrated by a comparison of computer cases I-3 and IV in Plate No. 7. An increase in scale from 1 to 1.5 increases displacements by F^2 or a factor of 2.25. Both horizontal and vertical strain increase directly with F and the displacement and strain ratios remain constant.

3.32 Influence of Embankment Height, h. If the embankment height increased with the same side slopes (b/h constant) and with the foundation depth constant, the settlement, vertical strain and horizontal displacement would increase approximately in direct proportion to the increase in h. However, the maximum horizontal strain remains approximately constant (Plate No. 12, Sept. 1960 report). This is because the increased horizontal strain produced by the larger vertical load is roughly balanced by the fact that the foundation layer is becoming thinner with respect to the embankment width.

3.33 Influence of Embankment Width, b. If the side slopes are flattened with an increase of the base width, the maximum settlement and vertical strain remain essentially constant. The maximum horizontal unit strain decreases with b^2, approximately, (Plate No. 12, Sept. 1960 report). The maximum horizontal displacement decreases with b, approximately. However, in the ordinary range of cases the base width is between 5 and 7 times the height and thus the effect of this parameter on the maximum horizontal strain is in the range of $5^2/7^2$ or only 1 to 2.

3.34 Influence of Foundation Depth, d. If the thickness of the compressible foundation increases, the maximum settlement increases in approximately direct proportion and the horizontal displacement increases approximately with $d^{2/3}$. This is illustrated by the computer cases in Plates Nos. 6 and 10. The vertical strain remains approximately constant, while the horizontal strain increases with $d^{2/3}$. The strain ratio increases with $d^{2/3}$, approximately, but the displacement ratio decreases with $d^{1/3}$. Typical ratios of b to d for the cases studied range from 4 to 12 and corresponding values of the strain ratio from the upper panel of Plate No. 10 are about 0.2 to 0.35. The displacement ratio in the same range varies from 0.3 to 0.2.

3.35 Influence of Embankment Weight, . If all geometric factors remain constant but the embankment material becomes denser, all displacements and strains increase in direct proportion to the unit weight but both the displacement and strain ratios remain constant.

3.36 Influence of Foundation Elastic Modulus, E_f. The influence of changing the foundation modulus while holding the embankment modulus constant is illustrated by the computer cases in Plates Nos. 4, 5 and 9. The parameter chosen is E_e/E_f since this provides a simpler relationship. Both displacement and strain values increase in approximately direct proportion to E_e/E_f, but the displacement and strain ratios remain nearly constant. The value of E_e/E_f for the ordinary cases probably ranges from about 1/2 to 1-1/2 and the strain ratio is 0.3 in this range, while the displacement ratio is 0.25.

3.37 Influence of Poisson's Ratio, ν. The effect of varying Poisson's ratio for embankment and foundation is illustrated by the computer cases in Plates Nos. 3 and 8. Both maximum settlement and vertical strain decrease in roughly direct proportion to Poisson's ratio in the probable range of interest from 0.2 to 0.4. In the same range the horizontal displacement and strain increases roughly in direct proportion to Poisson's ratio. Both displacement ratio and strain ratio increase approximately with ν^2. Thus the strain ratio increases from 0.1 to 0.4 and the displacement ratio from 0.08 to 0.36 in the range of ν from 0.2 to 0.4. The effect of Poisson's ratio is confirmed in part by the correction factor which relates variations in Poisson's ratio with change in displacement components within an embankment, as given by Woodward in Reference 14 and shown by dashed lines on Plate No. 8.

3.4 Conclusions from the Analysis

The computer solutions of the finite element analysis, supplemented by data in the September 1960 report, lead to the following conclusions on theoretical factors influencing interface displacement:

1. The analysis of the September 1960 report for Poisson's ratio of 0.25 on Plate No. 2 yields slightly higher values of the maximum strain ratio than the computer solutions for ν 0.35, whereas the reverse order would be expected. A comparison of results from the two analyses is shown in the upper panel of Plate No. 10. The difference possibly is the result of the assumption of triangular symmetrical shear in the earlier analysis, probably with a maximum shear stress which is too high and positioned too near to the dam axis. Nevertheless, the general effect of the independent variables is similar in the two methods of analysis.

2. The computer solutions yield a horizontal displacement diagram which peaks at a location 55 to 65 per cent of the half-base width of the dam outward from the axis. The ratio of maximum displacement to maximum settlement is a less sensitive parameter than the strain ratio. For intermediate values of Poisson's ratio, the displacement ratio is generally found in the range of 0.2 to 0.3.

3. The ratio of maximum horizontal to maximum vertical strain lies in the range of 0.2 to 0.35 for an intermediate value of Poisson's ratio. However, the influence of Poisson's ratio in the range of ν from 0.2 to 0.4 is critical, yielding values of strain ratio from 0.1 to 0.4. This variation covers practically the entire range of strains in the embankments under observation. It appears likely that the more subtle influences of elastic modulus such as anisotropy, non-linearity, variability with depth, cannot be as important as the Poisson's ratio effect. The upper panel of Plate No. 8 illustrates the very rapid increase in the strain ratio for Poisson's ratio above 0.4. This trend is confirmed independently by Woodward's analysis of a dam in Reference 14 which shows both displacement and strain ratios of about 0.75 at Poisson's ratio value of 0.45.

3.5 Factors Influencing Poisson's Ratio

In the elastic model used in these analyses Poisson's ratio appears to exercise a decisive effect on the horizontal strains under the embankment load. This section considers the parameters which influence its apparent value. The three principal factors are:

1. The fundamental effective angle of shearing resistance of the soil;

2. The degree of mobilization of shear strength under the embankment loading;

3. The tendency of the soil to consolidate or dilate under the applied shear stresses.

The role of the fundamental angle of shearing resistance or the effective friction angle, $\bar{\phi}$, is indicated in Plate No. 11. The at-rest pressure ratio, K_o, is related to $\bar{\phi}$ by the following expression which has been verified by research testing:

$$K_o = 1 - \sin \bar{\phi}$$

In a simple elastic medium K is a function of Poisson's ratio, as follows:

$$K_o = \frac{\mu}{1 \quad \mu}$$

Although these equations are derived from entirely different approaches, they indicate that there is some tendency for the value of Poisson's ratio to decrease as the basic friction angle increases, as illustrated on Plate No. 11. Thus, a fine grained plastic soil with $\bar{\phi}$ between 20° and 25° would exhibit a μ value in the range of 0.35 to 0.4. A coarse grained soil with angular particles might perform with μ between 0.25 and 0.3.

The role of the last two factors listed above is interconnected. That is, the extent of consolidation or dilation during shear depends on the degree of mobilization of shear strength. In almost any ordinary situation an earth dam would require a foundation of substantially overconsolidated soils which tend to consolidate under low load but dilate during shear. The higher the ratio of applied shear stress to peak shear strength, the greater would be the amount of dilation and the higher the value of Poisson's ratio evidenced at the end of the load application. While little reliable information is available from soil testing on Poisson's ratio, it is well established that the Poisson's ratio on dense crystalline rock typically increases from about 0.15 at the start of shear to 0.35 at failure as the change of volume passes from an initial decrease to a tendency for expansion.

It may be concluded tentatively that among likely earth dam foundation materials the highest value of Poisson's ratio would be exhibited by an overconsolidated plastic clay where the maximum embankment pressures are approaching the plastic stress value described in Section 2.2. The material with minimum Poisson's ratio possibly would comprise broadly graded, coarse-grained soils with angular particles where embankment pressures are substantially below the plastic stress limit. This implies that the lowest Poisson's ratio would be associated with the least compressible soil. While this may be the case for soils suitable for an earth dam foundation, it is not generally true. For example, a low Poisson's ratio would apply to the situation where a properly designed sand drain installation is used to stabilize highly compressible soils. In that case if the embankment load is added slowly to allow a gain in strength of the compressible soils, the horizontal movements can be quite small compared to the large vertical settlement.

The determination of Poisson's ratio is essentially impracticable in a modest program of production testing for small earth dams. Tentative methods for estimating its value from ordinary tests are described in the recommendations of Chapter 7.

4. -FIELD OBSERVATION PROGRAM FOR SOIL CONSERVATION SERVICE DAMS

4.1 Background of the Observation Program

A systematic program for observation of movements of articulated conduits beneath SCS earth dams was initiated with engineering memorandum SCS-45 dated September 11, 1961. That memorandum outlined procedures to be undertaken during design studies and field measurements to be made in order to assemble data which would permit refinement of tentative methods for estimating conduit movement. Sites were selected for study which were expected to involve significant consolidation of foundation soils and consequent elongation of the conduit. It was necessary that the conduit would be accessible for measurements after construction had been completed. Conduits included both reinforced concrete pipe and monolithic cast-in-place reinforced concrete boxes of rectangular cross section. In the majority of the 20 cases studied for this report the requirements set forth in memorandum SCS-45 were satisfied. However, a number of cases for which laboratory tests had indicated significant susboil compressibility actually produced small movements and posed difficulties in evaluation since the inherent irregularities in measurements then became relatively important.

4.2 Field and Laboratory Investigations

At each site selected for observation three borings were made and logged in detail along the conduit centerline before construction, one near the upstream toe of the embankment, one near the downstream toe and one at the embankment axis. Ordinary split spoon samples and undisturbed samples were taken in these borings for laboratory examination and testing. Undisturbed samples or sack samples of backfill beneath and adjacent to the conduit were obtained for testing. Values of in situ density, moisture content, Atterberg limits, and gradation were determined for selected samples. Conventional one-dimensional confined consolidation tests and triaxial shear tests were performed in the SCS Lincoln, Nebraska laboratory. Dry density and moisture content of embankment soils were determined at various locations in or adjacent to the vertical plane through the centerline of the outlet pipe. The test data were utilized to estimate the embankment safety factor, maximum foundation consolidation, and from this value, the maximum joint extension.

The structures which were studied and documented in this manner take on the status of test embankments and provide an amount of data which is much more detailed than usually is obtained for structures of their size. In most cases the field conditions were summarized by SCS personnel on a natural scale cross section of the embankment at the conduit location which included boring information without interpolating the subsoil stratification.

4.3 Conduit Observations

Reference points were placed along the conduit invert at about 1-1/2 inches from the ends of each pipe section, consisting of metal stove bolts set in lead expansion plugs in holes drilled in the conduit. These served as the basic measuring points for determining settlement and horizontal extension of the conduit. Reference points were installed by the contractor before earth fill had been placed above the conduit and an initial set of readings was made with the conduit unloaded and for each approximately 10-foot increment of embankment height above the conduit until the embankment was complete. At the same time each joint gap in the interior of the pipe was measured at four points, top and bottom of the pipe and at each side of the spring line, and the relative vertical and horizontal displacement at each joint at right angles to the axis was recorded. In the first year after completion of the dam, readings generally were made at three month intervals and yearly thereafter. Readings of settlement were taken to the nearest one-hundredth of a foot and horizontal distances and joint gaps to the nearest one-sixteenth of an inch. Measurements were recorded on Form No. SCS-214, which noted elevations of the embankment at the date of observations and included remarks on the condition of the conduit.

Taping both the long and short distance between reference bolts permitted determination of the elongation of the pipe sections between joints whether due to elastic strain or to cracking. However, the average of the four gap measurements at each joint provides a more accurate indication of the actual joint extension. The single measurement of joint opening at the reference bolts is distorted by the effect of rotation of the joint.

4.4 Cross Section at the Conduit Location

The field observations are summarized herein on a series of twenty drawings numbered from 1 to 20 to correspond with the assigned case history numbers. These drawings show a cross section of the embankment and foundation at natural scale taken in the plane of the conduit approximately at right angles to the axis of the dam. In the cases of dams studied herein, the zoning of the embankment between core and shells is not clear-cut and no formal zoning was prescribed or is depicted on the drawings. The original ground line, the base of excavation of the conduit trench, the cross section of the dam cut-off trench are plotted on the drawing. The section shows the top and bottom of the interior of the conduit at its constructed position, indicating length of cutoff collars and an outline of the intake structure. Borings taken at the conduit location are plotted with the Unified Classification of the materials encountered and certain identification test data. The principal subsurface strata interpreted from the boring data are drawn below the embankment and the apparent base of the compressible foundation soils is indicated.

4.5 Presentation of Conduit Observations

The following four movement components are plotted on scales below the embankment cross section:

1. Settlement of invert of conduit pipe;
2. Total horizontal movement of the conduit outward, upstream or downstream from the center of the embankment;
3. Horizontal opening at each conduit joint;
4. Rotation at each conduit joint.

Procedures for computing and plotting the movement components are described in the following subsections. Unloaded zero readings of the conduit in its as-placed position are available and all plotted data represent net values of movements. It should be recognized that the pipe is laid with some opening at the joint between the butting interior edges of the pipe and with some amount of rotation from a perfectly level or smoothly cambered position. The cambered position of the pipe means that the pipe sections are laid with some small amount of angle change between sections. Thus, it is of prime importance to record the unloaded position of the pipe, and in this respect the observations utilized for the September 1960 report are deficient since they do not distinguish these irregularities in the pipe's placed position.

For each of the movement components, two sets of readings were plotted and noted by symbol in Drawings Nos. 1 to 20; the movement obtained at the reading date closest after the date of completion of the embankment and the movement at the most recent reading date. The data listed in Table No. 5 include the total time in months that the structure has been under observation, the length of the observation period during construction and the length of the observation period following construction.

4.51 Settlement of Conduit. The observations provide a series of readings of elevations on the reference bolts taken from a bench mark set prior to construction. Since two bolts were placed at each joint, one at either side of the gap, it was possible to obtain accurate settlement profiles by averaging the two readings at a joint for each date. Settlements are plotted in the middle panel of the Drawings Nos. 1 through 20 in feet downward from the zero line.

4.52 Pipe Joint Movement. Pipe joint movements are plotted in shaded symbol in the lowest panel of the drawings with a value noted for each joint, generally starting with Joint No. 1 at the connection between the outlet pipe and the intake riser. Joint movement is plotted as inches of opening downward from the zero line or inches of closing upward from the zero line. The movement value is obtained by averaging the four joint gap readings for the date of observation. The joint gap measurements are connected with straight line segments in order to visualize the continuity of the readings although there is no physical connection between them. The joint gap observations tend to be somewhat erratic since they are influenced by the degree of tightness of the joint at laying and the position of the cut-off collars.

4.53 Total Horizontal Movement. Total horizontal movement in inches is plotted by shaded symbols in the middle panel of the drawings. For the plotting of total movements outward from the center of the dam, the location of zero movement cannot be established with certainty and the location of the joint with maximum settlement was arbitrarily assumed for this point. The total horizontal movement outward, upstream and downstream from this point was generally obtained by adding the joint gap readings plotted in the lower panel. However, in certain instances where cracking of the pipe between joints was noted on the observation form or where measureable and consistent lengthening was recorded between reference bolts within the pipe section itself, the values of elongation of the pipe section have been added. The only value that is reasonably certain is the total magnitude of outward displacement between the upstream and downstream limits of extension.

4.54 Joint Rotation. The values of joint rotation in radians are plotted in the lowest panel of the drawings with rotation convex upward plotted upward from the zero line and rotation concave upward plotted below the line. Joint rotation values are computed from the slope of the settlement profile as the angle change in radians between two adjacent pipe sections produced by settlement. Settlement readings alone are utilized to compute rotation and the apparent flexing of the joint indicated by the difference in the four joint gap readings are not considered. The values of joint rotation are relatively erratic because of their sensitivity to irregularities in the settlement profile and the maximum values listed in Table No. 5 are generally taken near the point where settlement is maximum and rotation is concave upward.

5. -REVIEW OF DAMS UNDER OBSERVATION

5.1 Introduction

Brief descriptions of site conditions and measured displacements at the twenty SCS earth dams under observation are presented in this chapter. The first thirteen cases are numbered in the sequence used in the Cappleman study. The seven highest numbered cases, 14 through 20, were not included in Cappleman's review. Cross sections shown on Drawings Nos. 1 through 20 are supplemented by the twenty sheets of Table No. 4, each one of which provides quantitative information on embankment and foundation characteristics. Data included are average values of natural moisture content, Atterberg limits, dry density, total density, void ratio and gradation characteristics for embankment soils and for the principal foundation strata distinguished on the cross sections. In each case the results of soil tests performed at the SCS Lincoln laboratory were utilized by SCS engineers to compute maximum settlement of the conduit. Maximum horizontal strain was estimated in the design study using the computed settlement and the tentative correlation of Plate No. 29 of the September 1960 report. Since the practicability of predicting horizontal movements is the crux of this study, these estimates made during design are discussed in the case histories.

5.2 General Site Conditions

Almost all of the twenty earth dams are founded on clayey or silty subsoils. The sites are concentrated in the southern middle west but include scattered locations in the north, east and southeast. A total of twelve states are represented. Assuming that site geology offers one method of distinguishing the performance of the dams, the sites have been grouped in the following four geological categories:

5.21 Geological Category A comprises foundation materials reworked by stream action from residual soils derived from sedimentary rocks, primarily shales. Eight of the nine cases in this group are in Permian or Pennsylvanian redbeds of Oklahoma and Texas. The residual soils are generally silts and clays of low to moderate plasticity and materials reworked from them exhibit a certain amount of stratification and interlensed sandy material due to their alluvial origin. The transition from overburden to underlying rock is generally abrupt.

5.22 Geological Category B includes similar reworked residual soils derived from parent sediments of Tertiary or Cretaceous age. These include three cases from Texas, Tennessee and Mississippi. The subsoils are generally sandier than those in Category A and they are distinguished by the fact that no abrupt transition is to be found between overburden and the parent sediments which are themselves merely stiff or compact soils in engineering terms.

5.23 Geological Category C comprises alluvium and residual soils originating in the weathering of crystalline bedrock and includes two cases in South Carolina and Georgia. Overburden materials range from sand to clay of moderate plasticity.

5.24 Geological Category D includes six sites in glacial terrain. Two of these in Iowa and Kansas contain loess or reworked loess over glacial till. Three sites in North Dakota and Kansas comprise clayey alluvium or colluvial materials over till. The sixth site in Massachusetts is underlain by varved glacial lake silt and clay on till.

5.3 Case Histories

Subsections 5.301 to 5.320 review briefly the basic information of Table No. 4, the observations of Table No. 5 and the interpreted data of Table No. 6. The following parameters are referred to and are restated for convenience:

"Computed settlement" was determined by SCS engineers from laboratory consolidation test data.

"Estimated horizontal strain" was obtained during design from computed settlement using the relationship of Plate No. 29 of the September 1960 report, a portion of which is reproduced as Plate No. 12.

"Observed strain ratio" is the ratio of maximum horizontal strain obtained from the greatest observed pipe joint opening to maximum vertical strain obtained from observed settlement.

"Theoretical strain ratio" is determined from Plate No. 29 of the September 1960 Report, using the b/d and b/h values interpreted from the embankment cross section, and is not related to the computations made during design.

"Foundation stress Ratio" is a parameter introduced by Cappleman and equals:

$$\frac{2pd}{sb}$$

where s = shear strength in the weakest upper portion of the foundation at completion of construction.

It represents the degree to which the embankment loading approaches the limit of elastic action defined by Jurgenson.

The observed and theoretical strain ratio are compared by plotting the results from each case on Plate No. 12. The overall quality of the comparison is discussed in Chapter 6. The value of computed settlement is subject to various interpretations and qualifications, and while it is mentioned in the following case histories it is not listed in the tables. In estimating probable foundation shear strength at the completion of the embankment, the test values of effective strength parameters were considered with the probable degree of consolidation at the end of construction. The equivalent vertical pressure at the elastic limit of performance, as defined by Jurgenson, is given in the last line of Table No. 6. The foundation stress ratio equals the actual maximum applied embankment pressure divided by this value. A foundation stress ratio of one or more indicates that plastic conditions are present at some point in the foundation.

The geometric parameters b, h and d for each embankment and its foundation are listed in Table No. 4. The estimated top of the incompressible underlying stratum is indicated by a rock line symbol on the embankment cross sections, even though the hard material is not necessarily bedrock. In computing the b/h and b/d ratios in Table No. 6, the b, h and d values of Table No. 4 are adjusted where believed necessary to reflect average conditions on the cross section.

5.301 Upper Black Bear Creek, No. 71. The site is in geological category A with silt and clay alluvium overlying shale and sandstone redbeds. Subsoils probably are overconsolidated to 2.5 to 3 ksf in excess of overburden but not to the magnitude of the maximum embankment load. Settlement computed during design studies was 1.25 feet, which compares closely with the observed maximum of 1.3 feet. Because wide berms were assumed on both slopes in design, the predicted strain ratio was 0.11, compared to the observed value of 0.19. The theoretical strain ratio for actual embankment and foundation dimensions equals 0.16. This is one of the most compressible foundation materials in the twenty cases and the foundation stress ratio is comparatively high, possibly nearly one. The pattern of horizontal displacements is typical of many structures, that is, there is a relatively narrow central tension zone with little compression upstream of this zone, in contradiction to the theoretical trend illustrated in Plates Nos. 3 to 7 which, of course, do not include the effect of berms. Because of the presence of the wide berm on the downstream slope, compression occurs where the embankment slope enters the berm. An unusual amount of cracking was noted at the pipe joints which apparently is due to joint rotation under the substantial settlements.

5.302 Powdermill Brook. This is the most unusual foundation materials in the twenty cases, consisting of a fairly thick deposit of glacial lake varved silt and clay in geological category D. The site is typical of narrow stream valleys in southwestern Massachusetts and northwestern Connecticut which were damned by moraines that formed sedimentation basins for fines carried in glacial melt waters. Atterberg limit values are highly variable from varve to varve. The average water content is higher than the average liquid limit, but in varved material this does not carry the usual implication of lightly consolidated material. Preconsolidation pressures were carefully determined in SCS laboratory tests and it was concluded that the varved material was overconsolidated to approximately the full embankment weight. Predicted maximum settlement was 0.93 feet using Δe taken directly from the e-log p curves. If a straight-line recompression index of 0.04 had been utilized, the computed settlement would have been close to the 0.8 feet maximum value observed. The observed strain ratio of 0.20 equals the theoretical value exactly. Considering the substantial preconsolidation, the foundation stress ratio is below one and elastic conditions probably obtain throughout.

5.303 Mill-Picayune Creek. Foundation materials fall in glacial category D, comprising Wisconsin loess underlain by reworked loess of the Sappa formation over till. The subsoils probably were preconsolidated to some extent but measured settlements are the greatest of any case under observation and subsoil compressibility the highest. The standard conduit observation program was supplemented by installing a series of settlement plates along the conduit axis at the base of the conduit trench excavation before placing backfill. The plate at the base of the embankment cutoff trench at the intersection with the conduit trench measured 1.02 feet of total settlement at the time when the conduit invert directly above had settled 1.71 feet. That is, there was 0.69 feet of compression in eighteen feet of fill and backfill above the plate and 1.02 feet compression in 38 feet thickness of foundation soils. In this case the strain in the natural foundation materials amounted to 2.7% compared to 3.8% in the fill and backfill, implying that the embankment material is about two-thirds as stiff as the foundation, a condition which may have resulted from the comparatively wet placement of the fill. It should be recognized that the presence of the backfilled conduit trench beneath the dam does not necessarily permit the conduit to move in a manner appreciably different from a parallel line in natural soils just outside of the trench. The stiffness of the overlying embankment will force a fairly smooth settlement pattern along lines at the top of the foundation taken parallel to the dam axis and crossing the conduit location. Because of the modest preconsolidation, it is probable that the limit of elastic action was approached by the embankment loading. The design estimate of total settlement and maximum joint opening was approximately 15% larger than observed values. The observed strain ratio equals 0.28 compared to a theoretical value of 0.35. It is interesting to note that in this case the joint opening at the end of construction was 94% of that finally observed, but the settlement at end of construction was only 73% of the final. This could be the result of saturation by reservoir seepage, which increased settlement of the loessial soils without a matching increment of horizontal strain.

5.304 Huff Creek No. 2A. This site is in geological category C with sandy alluvium reworked from residual soils overlying a deep profile of weathering in schistose bedrock. Water contents within the upper portion of the residual soil, Stratum 2, on the cross section, are unusually high and its in situ density is particularly low. Settlement was predicted during design as 1.4 to 1.5 feet, making allowance for hard layers within the residual soil. This is compared to the observed maximum of 1.6 feet. However, using Plate No. 29 of the September 1960 report, a maximum horizontal strain of 0.01 was estimated compared to the actual maximum of 0.0016. The observed strain ratio was 0.04 compared to a theoretical value for actual embankment and foundation dimensions of 0.27. This is the poorest correlation in the cases under observation. It is probable that the upper sandy alluvium is performing as a cohesionless material with a fairly high friction angle and serves as a stiff mat over the softer upper portion of the residual soil. In such a situation the settlements may be caused primarily by compression in Stratum 2 but horizontal movements originating in Stratum 2 are restrained by stiff layers at its top and bottom. As a result the foundation as a unit performs as if its Poisson's ratio is unusually low.

5.305 Big Wewoka Creek No. 39. Foundation materials consist of silt and clay reworked from soil residual from Pennsylvanian shales. The observations nearest after completion of the embankment were taken five months after construction. Probably for this reason an unusually large proportion of the finally observed displacement was measured at the time of completion of the embankment. Foundation soils probably are overconsolidated to the magnitude of the maximum embankment load. Computed settlements of 1.4 feet compare to the observed maximum of 0.4 feet. The predicted maximum horizontal strain was 0.016 versus the observed maximum value of 0.003. In this case a straight-line compression index of about 0.03 probably would have given fairly realistic strain values. The observed strain ratio of 0.31 compares well with the theoretical value of 0.35 for actual embankment and foundation dimensions. This is the next to highest observed strain ratio of the twenty cases. The comparatively high strain ratio implies that the foundation stress ratio is high, possibly near to one, as a consequence of the low value of b/d.

5.306 Cypress Creek No. 8. This is an example of a geological setting in category B wherein alluvial clay, silt and sand reworked from Eocene coastal plain materials overlie the parent dense clayey sands. The entire foundation probably is overconsolidated to the full extent of the embankment load. Settlements were predicted as a maximum of 0.5 feet with the accompanying horizontal strain of 0.007. In fact the observed settlements were one-third of this value and the maximum horizontal strain about one-quarter. The observed strain ratio is 0.16 compared to a theoretical value of 0.24. It is likely that the foundation stress ratio is low, equal to about 0.5, and as a result the effective Poisson's ratio is also comparatively low.

5.307. Upper Washita River No. 9. The site is in geological category B with reworked weathered products of the Tertiary Ogallala formation overlying the dense sands of the parent material. The subsoils undoubtedly are overconsolidated to the full extent of the embankment load. The top of hard materials is not clearly defined since the alluvium reworked from the Ogallala grades into the parent material without an abrupt discontinuity. In the design studies settlement was computed as 1.0 feet, allowing for some preconsolidation of the subsoils, compared to an observed maximum value of 0.4 feet. Maximum predicted horizontal strain was 0.012 compared to an observed value of 0.003. The observed strain ratio of 0.24 compares closely with theoretical strain ratio of 0.26 for the actual embankment and foundation dimensions.

5.308. Kickapoo-Sandy Creek No. BJ-1. This site is fairly typical of geological category A except that the alluvium is distinctly more plastic than in other cases. The pattern of joint openings is particularly unusual since essentially uniform extension was observed in the latest survey throughout the conduit. On the other hand, negligible joint opening and practically zero total horizontal movement was observed at the end of construction. An almost uniform increase in joint opening equal to 0.3 to 0.4 inches was measured in the period of 40 months following construction. This extension occurred gradually and continuously in the postconstruction period after the date when open water was first noted in the reservoir. However, reservoir water levels recorded on the observation forms have practically never risen above the spillway crest. In the same period the pipe sections themselves shortened throughout the entire length of the conduit but the measured shortening along the line of reference points at the invert was smaller than joint openings measured in the same period. A shrinkage of the pipe sections resulting from some peculiar circumstance in their manufacture or contraction of the pipe sections as a result of flow of cold reservoir water might have been responsible. Settlement was predicted during design as 1.0 feet compared to the observed maximum of 0.3 feet. Maximum horizontal strain of 0.01 was estimated compared to an observed value of 0.004. Surprisingly, the maximum observed strain ratio of 0.36 agrees almost precisely with the theoretical strain ratio 0.37 for the actual cross section dimensions. This case is distinguished by a low b/d ratio which tends to produce a relatively high foundation stress ratio and a relatively high strain ratio. The observed strain ratio is the greatest of any of the twenty cases under observation but this result must be considered questionable because of the highly unusual pattern of joint openings.

5.309 Quartermaster Creek No. 1. This is an example of geological category A which is distinguished by the condition that the most compressible subsoils appear to be the lower portion of the overburden. The foundation materials consist of an upper silty alluvium and a lower clayey alluvial stratum derived from soils weathered from underlying Permian redbeds. The lower alluvium has water contents above the plastic limit with sampler penetration resistance as low as 2 to 5 blows per foot. Maximum settlement estimated in design was 1.4 feet with a maximum horizontal strain of 0.016. These are to be compared with an observed maximum settlement of 1.2 feet and horizontal strain of 0.005. The observed strain ratio of 0.16 is only one-half of the theoretical strain ratio for the actual cross section dimensions. The reason for this discrepancy may be similar to that for Case No. 4, Huff Creek, wherein a stiffer upper layer tends to restrain horizontal movements accompanying compression of the lower softer stratum.

5.310 Tobesofkee Creek No. 70. The site is in geological category C with a thin cover of reworked residual soils grading into the parent schistose gneiss of the Piedmont. The maximum thickness of compressible materials beneath the conduit is only about 15 feet and the b/d ratio equals 20. Settlements computed during design totaled 1.1 feet compared to maximum observed value of 0.5 feet. The computed maximum horizontal strain of 0.009 is compared to the measured value of 0.0013. An unusual feature of this case is the regular and consistent joint compression observed under the upstream slope. This may result from the fact that the reservoir was filled to the principal spillway crest or higher throughout the later period of observations, applying a compressive force on the end of the steep slope of the upstream berm. The maximum observed strain ratio beneath the axis of the dam was 0.04 compared to a theoretical strain ratio of 0.12 for the actual cross section dimensions. This appears to be an additional example that a thin compressible foundation zone will deform as if the effective value of Poisson's ratio is lower than the median value assumed in the theoretical analysis.

5.311. White Clay, Brewery, Whiskey Creek No. 6. The site is in geological category D, situated in moderately to steeply rolling hills of Kansan till with a thin deposit of reworked Loveland Loess in the stream bottom underlain by Kansan and Nebraskan till sheets. The conduit consists of cast-in-place concrete pipe in 20-foot lengths of square cross section. The case is distinguished by the fact that the conduit is placed well below the original ground line in a trench which was excavated to below the top of the underlying Kansan till. This placement condition tends to minimize both conduit settlement and extension. The subsoils probably are preconsolidated in excess of the embankment load. Maximum settlement predicted during design was 0.8 feet compared to the observed value of 0.15 feet. The observed maximum strain ratio was 0.09 compared to a theoretical value of 0.13.

Although the measurements are not definite, shrinkage during curing of the conduit sections probably increased joint openings, improving the correlation between measured and theoretical strain ratio.

5.312 Upper Washita River No. 57. This is an additional case in the geological category A with silty, sandy and clayey alluvium in the stream valley derived from weathered products of the underlying shale. This is the highest dam studied, with a maximum height above original ground at the conduit equal to 50 feet and a maximum embankment loading of 6.4 kips per square foot. Settlement predicted during design amounted to 1.2 feet compared to an observed value of 1.0 feet. The observed strain ratio of 0.08 compares with the theoretical value of 0.14 for the actual cross section dimensions. Considering the high b/d ratio of 17, this is a relatively favorable agreement. The case is noteworthy in that only about 2/3 of the total settlement and an even smaller proportion of the maximum joint opening was observed at the end of construction. The ground water table at the time of the original field exploration was relatively low in the borings and it is possible that wetting of the subsoils by reservoir seepage since the end of construction has increased the proportion of post-construction movements.

5.313 Upper Red Rock Creek No. 33. This site is in geological category A with alluvial clays derived from soils weathered from underlying redbeds. The foundation soils may be preconsolidated to the full extent of the embankment loading. The apparent heave in the settlement profile at either end of the conduit at the date of the latest survey is in doubt because of possible disturbance to the reference points. Settlement estimated during design studies equaled 0.8 feet compared to an observed maximum of 0.26 feet. The predicted maximum horizontal strain was 0.01 compared to the observed value of 0.0024. The observed strain ratio of 0.17 is not greatly below the theoretical strain ratio of 0.24. Part of the discrepancy may result from the fact that the most clayey and compressible material responsible for the greater portion of the settlement is in the lowest overburden immediately above bedrock.

5.314 Bristows Creek No. 1. The site is in geological category A but differs from others in this group since the parent rock is Mississipian limestone. Overburden is derived from weathering of this limestone and consists of a mixture of angular silica fragments with clay wherein the proportion of coarse particles increases with depth. The upper 20 to 25 feet of the overburden is a sandy clay with liquid limits between 40 and 45 and water contents near to the plastic limit. Settlement computed during design was 1.1 feet compared to an observed maximum of 0.6 feet. Estimated maximum horizontal strain was 0.008 compared to an observed value of 0.0013. The observed strain ratio was only 0.06 compared to a theoretical value for actual foundation and embankment dimensions of 0.24. The possibility that the thickness of compressible foundations soils is substantially less than assumed for this analysis may contribute to the poor correlation. This case is unusual in that joint compression of substantial magnitude was observed throughout the conduit beneath the upstream slope. In this respect it is similar to Case No. 10, Tobesofkee Creek No. 70. The reservoir water level was raised shortly after construction and has been held up continuously through the later observations at a level near the principal stillway crest, which is high compared to the embankment crest. The water loading may be responsible for settlement observed at the upstream toe near the intake structure, which is almost one-third of the settlement at the dam axis. This balancing reservoir load may have contributed to the low observed strain ratio. The pattern of total horizontal movements is of normal shape downstream of the axis.

5.315 Porter's Creek No. 11. This site is an example of geological category B with alluvial and colluvial clay over Tertiary clay-shale. Water contents of the overburden near to the plastic limit indicate the clay is highly preconsolidated, probably much in excess of the maximum embankment loading. Field difficulties with the borings may have invalidated the blow counts determined from standard penetration tests. This case includes next to the lowest magnitude of observed settlement and joint opening. Maximum settlement of 0.76 feet was predicted during design, compared to an observed value of 0.14 feet. This over-prediction led to an anticipated maximum horizontal strain of 0.008 compared to the observed value of 0.0005. The observed strain ratio equals 0.10 compared to the theoretical value of 0.30. The b/h and b/d ratios are average in value and the foundation material is relatively uniform. The foundation stress ratio is probably low, perhaps near to one half, and the poor quality of the correlation of observed to theoretical values may be due to a low effective value of Poisson's ratio.

5.316 Tewaukon No. T1-A. This site is an example of glacial category D with alluvium derived from glacial till overlying weathered till. Settlement computed during design equaled 0.9 feet compared to an observed value of 0.3 feet. Predicted maximum horizontal strain was 0.005 compared to a measured maximum of 0.0015. The observed strain ratio equals 0.11 compared to a theoretical value of 0.19. This discrepancy undoubtedly is due to a low stress ratio in the foundation and a consequent low effective value of Poisson's ratio.

5.317 Tewaukon No. T-2. This is another example of glacial category D and may be compared directly with Tewaukon No. T1-A, except that the plastic surface soils derived from till and the weathered till are substantially thicker than at the Tewaukon No. T1-A site. The settlement profile has an unusual asymmetrical shape which may reflect the presence of deeper alluvium beneath the upstream slope at the point where the conduit trench excavation was carried to greater depths. The entire subsoil profile is probably overconsolidated in excess of the maximum embankment loading. Settlement computed during design was 1.0 feet compared to a measured maximum of 0.54 feet. The predicted maximum horizontal strain was 0.007 compared to an observed value of 0.0025. The observed strain ratio of 0.21 correlates with a theoretical value of 0.35 . This discrepancy may be the result of a comparatively low stress ratio in the foundation soils leading to a low effective value of Poisson's ratio.

5.318 Upper Wabash No. 1. This is another example of glacial terrain in geological Category D with alluvium derived from glacial till overlying till on dolomitic limestone. The overburden soils are probably overconsolidated in excess of maximum embankment loading. For this analysis the top of the rigid foundation was taken at the surface of the dolomitic limestone, but in fact it is probable that the lower half of the till is incompressible compared to the overlying soils. Settlement computed during design equaled 0.33 feet compared to the observed value of 0.09 feet, the smallest settlement of any case under observation. The estimated maximum horizontal strain was 0.004 compared to an observed value of 0.001. The conduit is a cast-in-place concrete box in 34 foot lengths with square coross-section four feet in dimension. The field observations did not include measurements of the change of length of the conduit sections, but it is likely that the horizontal strain determined from joint openings is partly due to shrinkage of the concrete in setting. The observed strain ratio of 0.28 agrees well with the theoretical values of 0.33. This agreement is invalidated by probable shrinkage strains in the pipe sections themselves which tend to increase the apparent joint opening.

5.319 Sugar Creek No. 41. This site is another example of geological category A, except that the parent rock is sandstone and the overburden is a non-plastic silty medium sand, loose in its upper portion and becoming more compact with depth. Maximum settlements computed during design equal 0.54 feet compared to an observed value of 0.79 feet. The estimated horizontal strain of 0.004 compares to an observed maximum value of 0.0024. The unusual condition of a computed settlement less than the observed value may result from the tendency of sand samples to become more compact during sampling, shipping and laboratory processing. The observed strain ratio of 0.15 is to be compared with the theoretical value of 0.33. The difference is probably the result of a low stress ratio in the foundation with a consequent comparatively low effective value of Poisson's ratio.

5.320 Kickapoo No. 4. This site is in geological category A, except that the parent shale is particularly shallow and the overlying alluvial clay is thin. Water contents in the clayey alluvium are generally below the plastic limit. Settlement estimated during design was 0.7 feet compared to an observed value of 0.14 feet. Maximum estimated horizontal strain was 0.04 compared to an observed value of 0.0005, the poorest comparison of any of the 20 cases. The observed strain ratio equals 0.04 compared to a theoretical value of 0.15. The difference is undoubtedly due to the relatively low stress rationin this foundation and a consequent low effective value of Poisson's ratio.

6. -DISCUSSION OF FIELD OBSERVATIONS

6.1 Introduction

The principal purpose of the evaluation of data in Chapter 5 has been to compare values of the maximum observed strain ratio with the theoretical strain ratio interpreted from Plate No. 29 of the September 1960 report. In addition, information on a number of other characteristics has been obtained from the field observations, including the pattern of total horizontal movement, the compressibility of various subsoils, the influence of foundation stress ratio on conduit movement, and the magnitude of joint rotation. This chapter considers each of these items and finally reviews Cappleman's independent analysis.

6.2 Statistical Evaluation of the Field Observations

Elements of the cross section geometry, observed deformations, and interpreted factors are summarized for the entire set of twenty cases in Table No. 7. Data are tabulated in statistical fashion, listing each value in the array for one case smaller, five cases smaller, median, five cases larger and one case larger. It should be noted that each parameter on a horizontal line in the table has been evaluated separately and that there is no consistent relation between cases vertically in the table. For example, the ratio b/h is analyzed as a separate factor for the twenty cases instead of dividing the b value by the h value listed in a particular column.

Table No. 7 indicates the quality of the correlation between observed and theoretical strain ratio. In only one case is the observed value larger than the theoretical. The median value of the ratio of observed to theoretical is 0.63 and five cases fall below 0.45. Thus, it appears that the suggested method set forth in Plate No. 29 of the 1960 report is an upper limit of performance for small dams. If the tolerable deviation of observed from theoretical values is taken as 30 per cent, then about one-half of the cases conform with reasonable accuracy to the theory. It should be recalled that this correlation is based on hindsight and complete knowledge of field conditions and does not consider the additional difficulties of prediction during design.

6.3 Factors Influencing Observed Strain Ratio

It became apparent in the study of case histories that the poorest correlation with theory is obtained in those cases where the foundation layer is relatively thin and stiff and observed horizontal movements are much less than theoretical values.

The effect of this extra foundation restraint is expressed roughly by the foundation stress ratio listed in Table No. 8. The relationship between the ratio of observed to theoretical strain ratios and foundation stress ratio is plotted in Plate No. 13. This shows that the best correlation obtains at the highest stress ratio where plastic action is approached.

The computer solutions were utilized to determine the effect of the various parameters on the theoretical strain ratio as listed in Table No. 3. They confirmed that the arrangement of geometric parameters h, d, and b used in the 1960 report was essentially correct but showed that the theoretical strain ratio from an elastic solution is greatly affected by the value of the Poisson's ratio. The foundation stress ratio probably combines the factors which influence the effective value of Poisson's ratio. These include not only the foundation stress level or the degree to which plastic conditions are approached, but also the effect of the horizontal restraint provided by a rigid layer at a shallow depth below the ground line.

Several other factors not considered in the analysis appear to affect the correlation between observed and theoretical strain ratio and these are noted by symbol in Plate No. 13. The use of cast-in-place pipe sections apparently increases the joint opening due to concrete shrinkage and thus increases the observed strain ratio. The presence of a reservoir load acting on the upstream slope for a considerable period of time tends to decrease the outward horizontal movement, decreasing the observed strain ratio. Where a relatively compressible foundation stratum is present below an upper, more resistant layer the observed strain ratio is decreased, undoubtedly as a result of the increased horizontal restraint acting on the compressible layer. None of these factors has been considered in the theoretical analysis. No consistent relation between the quality of the theoretical correlation and the general geologic category is apparent.

6.4 The Foundation Stress Ratio

The foundation stress ratio is only a rough measure of the limit of elastic action in the foundation. More precise analysis of the conduit deformation problem would require a better definition of the factors which control the equivalent Poisson's ratio in a particular situation. The probable limits of the foundation stress ratio can be rather easily defined, as illustrated by the following examples.

1. Upper value of stress ratio:

Assume clay foundation preconsolidated to $(2/3)p = P_c$, $s = 0.3P_c = 0.2p$ $b/d = 7.5$; at end of construction effective stress acting in foundation is only $2/3p$;

$$\text{Then, } \frac{2pd}{s\,b} = \frac{2p\ (1)}{.2p\ (7.5)} = 1.33$$

2. Lower value of stress ratio:

Assume clay foundation preconsolidated to $2p = P_c$, $s = 0.3P_c = .6p$, $d/b = 15$;

$$\text{Then, } \frac{2pd}{sb} = \frac{2p\ (1)}{.6p\ (15)} = 0.22$$

The computer solutions indicate that an observed strain ratio of 0.3, which is associated with the stress ratio of one or more, involves an effective Poisson's ratio of about 0.35. On the other hand, observed strain ratios less than 0.1 are associated with stress ratios less than 0.3, and, according to the computer solution, involve an equivalent Poisson's ratio between 0.1 and 0.2. It may be concluded that the quality of the theoretical correlation for these low dams is best for relatively soft and compressible foundation soils which are not preconsolidated greatly in excess of the embankment load.

6.5 Pattern of Horizontal Displacements

Maximum joint opening in Table No. 7 ranges from about 0.1 to 0.9 inches with half the observed cases falling between one-quarter and one-half inch. The ratio of maximum horizontal displacement outward from the axis to maximum settlement generally varies between about 0.1 and 0.3. Table No. 7 shows that the displacement ratio is close to the maximum strain ratio except in the highest range of values where extension of the pipe sections adds to the cumulative displacement. As noted in subsection 4.53, the value of the total horizontal movement plotted on Drawings Nos. 1 through 20 cannot be considered to be highly accurate. Measured elongations of the pipe sections themselves are based on only one set of taped observations and are therefore somewhat erratic. In addition, the zero point of horizontal displacements near the dam axis cannot be definitely established so that the dividing line between movements upstream and downstream is unknown. For this reason the horizontal displacement, L_h, is taken as one-half of the total observed horizontal extension.

It is noteworthy that the shape of the observed horizontal displacement diagram differs sharply from the theoretical shape plotted from computer solutions. In almost all cases the horizontal displacements upstream of the dam axis pass through a central zone of marked tensile strains followed by decreasing tensile strains without the definite compression beneath the shell of the embankment indicated by theory. Downstream of the axis the presence of compressive strains beneath the shell is more commonly observed. The reasons for the departure from theory may be as follows:

1. Such a pattern of readings suggests that plastic conditions may be developing beneath the upstream shell of the dam which would negate the buttressing effect of the shell.

2. It is possible that the pressure pipe joints, which generally have a rubber gasket bearing against steel surfaces, are wedged so tightly that further joint closing cannot occur and that the pipe cannot reflect the changes in strains in the surrounding soil.

3. The conduit slopes through the embankment from a high level at the upstream toe to a low level at the downstream toe. It is possible that the upstream portion of the conduit passes into the embankment zone in which tensile strains extend to the upstream face.

The validity of the first explanation is put in question by the generally low foundation stress ratio and the fact that the time rate of tensile strains, in all cases except No. 8, decreases sharply after construction. The second reason cannot be entirely responsible since in practically all cases some consistent joint closing has been observed, and occasionally this is the dominant pattern beneath the downstream slope. It is believed that the third explanation is the most likely, particularly when it is considered that outward pressures from the upstream shell act on the back of the intake structure which may tend to displace it upstream and drag the conduit with it. It must be concluded that the information on pattern of horizontal displacements in these small embankments cannot be utilized to evaluate the accuracy of the theoretical analyses.

6.6 Time Effects

Table No. 7 indicates that the typical length of time from initial measurement to end of construction is two months, while the typical interval from end of construction to the latest observations is 2-1/2 years. In the average case 85 per cent of the settlement finally observed and 70 per cent of the joint opening occured during construction. There is no ready explanation for the difference in those two values. It is possible that the flow of cold water from the reservoir tends to cause pipe shrinkage that adds to the post-construction joint opening. These data imply that in most cases the earth dam loading produces primarily recompression strains in the foundation with a high coefficient of consolidation that characterizes settlement in the recompression range. The fact that the time rate of movement decreases markedly after construction suggests that elastic conditions generally obtain. In most instances the increments of settlement or joint opening obtained from successive readings several years after construction are negligible, indicating that primary consolidation is essentially complete. The observations on Case No. 8 are an exception to this rule. However, as discussed in Section 5.308, the observed shortening of the pipe sections account for a significant amount of the measured joint opening and this example of possible plastic conditions must be largely discounted.

It should be recognized that much larger embankments than those under observation may produce drastically different time-displacement patterns. If foundation settlement occurs primarily within the range of virgin compression the coefficient of consolidation is much lower and settlements are delayed. In addition, the possibility of development of plastic conditions increases as the embankment height increases and this factor would increase the proportion of post-construction movements.

6.7 Use of Settlement Computations to Predict Joint Opening

The twenty cases illustrate the practicability of using production consolidation testing to determine maximum vertical strain and from this to estimate maximum horizontal strain. Three factors influence the quality of the final result:

1. The settlement computation and the selection of the depth value d must be reasonably accurate.

2. The selection of the effective base width of the dam and embankment height must be realistic.

3. The correlation between the theoretical strain ratio and the basic parameters must be reliable.

Errors in the second item should not be of significant magnitude. Deficiencies in the theoretical correlation may be remedied in part by use of the empirical diagram of Plate No. 13. Accuracy of the settlement prediction remains a major limitation.

In seven of the twenty cases the computed settlement was within 30 per cent of the observed maximum settlement. In two of these instances the observed settlement was slightly larger than the predicted value. This should be considered a result sufficiently accurate for the purpose. In the remaining 13 cases the observed settlement was less than 60 per cent of the computed value and in the least favorable cases the observed settlement was only about one-fifth of the predicted value. Invariably, observed settlements greater than about 0.7 feet were predicted satisfactorily. The greatest deficiency occurred in the case of the smallest observed values. Thus, the settlement analysis appears suitable for those cases where movements are appreciable but greatly overpredicts where settlements are small. The difficulty is compounded by the fact that it is these latter cases in which the theoretical correlation is excessively conservative, also by a factor as great as five to one. Thus, it is possible to make a prediction of joint opening which is so conservative as to be meaningless.

It is axiomatic that an accurate settlement prediction depends on a realistic determination of the preconsolidation stress of the foundation strata. No settlement computation should be attempted without first plotting the best estimate of preconsolidation stress from top to bottom of the layer. It should be recognized that it is practically impossible for a stable earth dam to be constructed on truly normally consolidated subsoils. In fact, the only truly normally consolidated soils probably are those currently being formed under open water. The preconsolidation stress should be assessed by every possible means, including geological evidence of the past existence of higher terrace levels in the valley, and correlations of consolidation stress with undrained strength.

In moderately to heavily overconsolidated soils it is generally unsatisfactory and excessively conservative to predict the recompression settlements by obtaining Δe from the initial curved portion of the e-log p test curve. Typical values of the straight-line recompression index ordinarily are equal to 1/8 to 1/12 of the initial water content taken as a decimal value. In heavily overconsolidated soils a far more realistic prediction of recompression settlements is obtained from such values rather than by utilizing the test curve directly in the recompression range.

6.8 Application of Theoretical Solutions

It must be recognized that analyses of movements within an earth dam's foundation requires a system which relates stresses, strains and displacements in all directions. The only available systems for computing such relations are based on elastic behavior of the elements in the dam and foundation. Thus, while the soils in the foundation may be only partially elastic, in the sense that strain and stress are directly proportional, and the embarkments themselves, if predominately granular soils, may be only distantly related to elastic behavior, elastic parameters must be found which produce strains and displacements comparable to those in soils. The basic elastic parameters are modulus of elasticity and Poisson's ratio. The problem is to find means for determining equivalent values for modulus of elasticity and Poisson's ratio that, in elastic analyses, will reproduce the actual soil deformations with satisfactory comparability.

The conventional modulus of elasticity determined by the vertical loading of cylindrical soil samples in unconfined or triaxial tests is not usually related to a value of Poisson's ratio. Three other types of tests, isotropic consolidation, confined consolidation and plane strain, yield a stress-strain modulus whose magnitude is a function of Poisson's ratio. The relation between the apparent modulus evidenced in each of these three tests to the "basic elastic modulus" (E_b) determined in unconfined compression or triaxial shear is listed in Plate No. 14. The ratios between E_b and the apparent moduli from these types of tests are plotted versus Poisson's ratio on Plate No. 15. While the procedure is untried, it may be possible to estimate the effective Poisson's ratio by comparing the vertical stress-strain modulus obtained from different test types. Perhaps the most promising comparison could be made between results from confined and isotropic consolidation, shown by the dashed curve in Plate No. 15.

The average foundation compressibility observed in the twenty cases is summarized in Table No. 8 as an apparent elastic modulus, taken equal to the maximum embankment pressure divided by the measured maximum vertical strain. Maximum strain equals maximum settlement divided by total thickness d, of the compressible foundation layers. These values generally range between 100 and 600 kips per sq. ft. and average 260 ksf. Values below about 160 ksf are associated with the larger observed settlements the more accurate settlement predictions and equivalent Poisson's ratio which may be as high as 0.3 to 0.35. These probably represent conditions where the maximum embankment loading exceeds the preconsolidation stress by some amount. Apparent E values higher than about 180 to 200 ksf probably represent conditions where the foundation soils are overconsolidated to the full magnitude of the embankment loading. These include 14 out of the 20 cases under observation.

The values of elastic modulus are not clearly distinguished by geologic category. For example, the glacial category D includes the lowest E in loess over till and the highest E in glacial till. The varved glacial lake silt and clay loaded in the recompression range exhibits the relatively high value of 480 ksf. As might be expected, the lowest modulus values are associated with materials having natural water content closest to the liquid limit.

6.9 Effect of Scale Factor

Section 3.31 noted that as the overall scale of the embankment and foundation increases the strain ratio theoretically remains constant. The observed cases suggest that this conclusion is not correct since as the scale of the section increased the foundation stress ratio tends to increase and this appears to be a factor of dominating importance. It would be expected that for higher earth dams the effective Poisson's ratio might reach or exceed 0.3 to 0.35, which is the probable value for the theoretical correlation of Plate No. 12, and it is entirely possible that the maximum strain ratio and maximum displacement ratio would be larger than the 0.45 maximum of Plate No. 12. There is some indication from much larger structures that the strain ratio and displacement ratio can be as much as 0.75 or even greater, indicating that the equivalent Poisson's ratio is in the range of 0.4 to 0.5. Thus, extrapolation of the observations in this study to much higher structures must be done with caution, considering the accompanying increase in equivalent Poisson's ratio.

6.10 Joint Rotation

In the September 1960 report the observed joint rotation was compared with the average slope of the settlement profile, taken as the settlement multiplied by the factor 2/b. In the twenty cases studied herein the average slope of the settlement profile generally ranges from 0.001 to 0.01 radians and the maximum joint rotation determined from the observed settlement varies from 0.003 to 0.02 radians. The ratio between measured joint rotation and average slope of the settlement profile lies between 1 and 3, compared to the range of 2 to 4 determined in the September 1960 report. The difference between these two sets of data is probably due to the fact that the observations in the September 1960 report did not eliminate irregularities in the placed position of the conduit. In the twenty cases studied herein the larger values of the ratio of observed joint rotation to average settlement slope generally occurs with the smaller value of settlement. Usually where settlement of structures exeeds one foot, the maximum joint rotation is less than two times the average slope of the settlement profile. Where settlement is less than 0.5 feet, the ratio of joint rotation to the average slope of the settlement profile exceeds two. Observations of joint rotation utilized have not included the occasionally large values which are measured near the outlet end due to movement of the support of the outlet section. Certain erratic individual rotation values which appear to be due to some peculiarity in the pipe bedding have been ignored. A fairly realistic estimate of the maximum joint rotation due to settlement can be obtained by using a multiplier of 1.5 to 2.5 times average settlement slope in cases where movements are expected to be appreciable.

6.11 Evaluation of the Cappleman Analysis

The Cappleman analysis presented in References 4 and 5 was based on the observed cases Nos. 1 through 13. Results of the study are summarized in Table No. 9. Maximum horizontal strain was obtained by averaging the measured joint opening at top and bottom of the pipe, without including joint measurements made at the spring line or the elongation of the pipe sections between joints. Consequently the pattern of individual joint openings obtained by Cappleman is somewhat more erratic than those determined herein and Cappleman utilized a method of averaging out the extreme joint openings beneath the dam axis. Nevertheless, his study indicates somewhat larger joint openings and larger observed strain ratio than obtained herein. His median value of observed strain ratio for the first thirteen cases is 0.26 which exactly equals the median value of the theoretical strain ratio derived from Plate No. 12. Cappleman then introduced the foundation stress ratio to explain variations of individual cases from the theoretical strain ratio.

Working from the basic data of September 1960 report, Cappleman combined the parameters controlling horizontal strain due to boundary shear stresses with the parameters controlling horizontal strain produced by triangular vertical loading to obtain the "foundation strain factor", equal to C_c pdh/b^2. Cappleman then devised a relationship between the maximum observed strain ratio plotted to a logarithmic scale versus the foundation strain factor plotted to an arithmetic scale and showed that the resulting correlation depends on the foundation stress ratio in each case. The importance of the foundation stress ratio has been strongly confirmed by the present study, and it is apparent that no rational analysis is possible without consideration of the shear stress level in the foundation.

Cappleman's study emphasizes the importance of the initial degree of saturation on the proportion of the total movement which takes place during construction. In several cases noted herein, particularly the loess of Case No. 3, post-construction movements may be increased by increased saturation from seepage. In general, however, the subsoils are not notably sensitive to the probable small change in the degree of saturation after construction.

In summary, our analysis of the case histories differs from Cappleman's in several respects:

1. Since we have averaged all of the available readings of joint opening, the array of values in the tension zone is more regular that Cappleman's and our individual maximum joint opening tends to be less than his.

2. The selection of parameters in Cappleman's foundation strain factor gives equal weight to the effect of vertical loading and boundary shear stresses, whereas for an intermediate value of equivalent Poisson's ratio the horizontal movements are almost exclusively due to the applied shear stresses.

3. Our evaluation of the foundation soils suggests that Cappleman's assigned shear strengths tend to be too low and that only the exceptional case actually approaches the limit of elastic action.

7. - CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

The principal feature of this study is the analysis of twenty low earth dams containing articulated conduits whose movements were observed by SCS during and after construction. Measured vertical and horizontal movements of the conduit pipe were expressed in terms of the ratio of maximum horizontal to vertical strain, the factor used to predict joint openings in the September 1960 report. These observations show that only those cases with the largest settlement and the thicker, softer and more compressible foundation materials correlate well with the theoretical analysis of the September 1960 report. The poorest correlation appears in those cases where the foundation layer is thin compared to the base width of the dam and is preconsolidated in excess of the maximum embankment loading. In these instances the foundation performs as if the equivalent Poisson's ratio is much lower than that assumed for the analysis of the September 1960 report.

7.2 Computer Analysis

Utilizing the finite element technique, a computer solution was devised for displacements of a simple symmetrical embankment on an elastic foundation to determine the theoretical effect of various parameters on settlement and horizontal movement. Because of the difficulties encountered in development and debugging of the program, it was not practicable to extend the computer program to consider details of the embankment geometry or variability of the foundation properties. The computer solutions generally confirmed the influence of the various parameters presented in the analysis of the September 1960 report. They indicated, however, that the symmetrical triangular shear loading of the September 1960 report is more severe than the typical condition developed during elastic deformation. The computer solutions show clearly the importance of the magnitude of the equivalent Poisson's ratio on the ratio of horizontal to vertical strains. They indicate that the cases with the highest observed horizontal movements exhibited an equivalent Poisson's ratio between 0.3 and 0.35.

While it was not possible to extend the computer solutions to cases of varied and more complicated embankment geometry and to cases of variable equivalent elastic properties in the embankment and in the foundation, it is believed that the basic solution developed can be so extended without undue difficulty. The major requirement will be enlarged computer capacity and a considerably increased expenditure of computer time for the solution of each case. We believe that such an effort would be well worthwhile to increase understanding of the displacements in larger and more complex embankments contructed on yielding foundations.

7.3 Importance of Foundation Stress Ratio

An important qualification to the basic theory was introduced by Cappleman, that is, the influence of the foundation stress ratio which expresses the degree to which stress state approaches the limit of elastic action. In the thirteen cases which he studied Cappleman utilized this factor to explain the variations from the basic theory of cases with the largest observed strain ratio. However, in our analysis it is the larger values of observed strain ratio which correspond most closely with the theory and the foundation stress ratio is utilized to explain those cases where the observed strain ratio is much lower than the theoretical value. The foundation stress ratio is only a crude indication of the onset of plastic action and it is probable that, if the computer solutions can be extended, a more realistic definition of the limit of plastic action can be devised. As an interim measure it is recommended that the foundation stress ratio be utilized in the manner shown in Plate No. 13 to modify the theoretical maximum strain ratio.

7.4 Prediction of Joint Opening During Design

Apart from difficulties in the theory, a basic problem in the prediction of maximum horizontal strain is the quality of the settlement analysis that can be made under production conditions. For the twenty cases studied, it is those with the largest observed settlement which were predicted accurately and it is these in which the theoretical solution is most nearly applicable. In the majority of cases a substantial overprediction of the settlement and the deficiency in the basic theory led to a large overestimate of joint opening. It is recommended that the settlement analysis concentrate attention on the evaluation of the probable preconsolidation condition determined from consolidation tests, but also utilizing geological evidence and data from undrained shear tests. If it can be established that the foundation is overconsolidated a nominal value of recompression index should be used in computing settlements, rather than to estimate Δe directly from the e-logp curve.

7.5 Further Investigations

For the low dams studies herein the maximum joint opening can be predicted accurately in those cases where settlement is relatively large and disjoining of the pipe could be a problem. The real difficulty is to eliminate those cases from consideration where movements will be insignificant. If this analysis is to be applied to substantially larger dams the possibility must be considered of increased values of equivalent Poisson's ratio. Improvement of the analysis could be achieved by further investigations in three areas:

1. Field observations similiar to those made for this study should be continued on selected larger dams. Exchange of information with other public agencies in this country and abroad working in this area would be particularly useful.

2. For increased understanding it probably will be necessary to greatly expand the theoretical data by the computer solution developed herein or by other similar methods of analysis. Information is needed on the stress distribution to better define the limit of plastic action. The effect of major changes in the embankment geometry on stresses and displacements should be determined.

3. Based on the theoretical stress distribution in the foundation it may be possible to estimate the equivalent Poisson's ratio from laboratory tests which duplicate these stress conditions. The procedure suggested herein for comparing the stress-strain modulus obtained from different types of tests to estimate Poisson's ratio might prove useful as an interim method.

4. Ultimately, correlations of observed displacements in larger embankments with expanded theoretical solutions for more realistic and complicated embankment geometry and foundation strength variations are needed to determine the best means for selecting suitable values for equivalent elastic parameters and, more importantly, to show the magnitudes of displacements and ratios of displacements that may be considered normal under the loads of dams on yielding foundations.

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TABLE NO. 2, SUMMARY OF COMPUTER ANALYSES

Embankment Values		Foundation Values		Embankment & Foundation Poisson's Ratio
Height, Feet	Modulus of Elasticity, ksf	Depth, Feet	Modulus of Elasticity, ksf	
40	100	40	100	0.2
40	100	40	100	0.3
40	100	40	100	0.35
40	100	40	100	0.4
40	100	40	100	0.45
40	100	40	25	0.35
40	100	40	50	0.35
40	100	40	200	0.35
40	100	40	400	0.35
40	100	20	100	0.35
40	100	60	100	0.35
60	100	60	100	0.35

neral assumptions for all cases:

1. Elastic conditions throughout and geometry as set forth in Section 2.2.
2. The embankment consists of a symmetrical triangle with side slopes of 3 horizontal to 1 vertical.

TABLE NO. 1, DEFINITIONS AND PARAMETERS RELATING TO CONDUIT MOVEMENTS

Definitions, Geometric Factors:

Height of embankment above top of foundation at axis of embankment in the plane of the conduit = h

Average thickness of the compressible foundation at conduit location = d

Width of the embankment at the conduit location between upstream and downstream toes = b

Length of individual section of the articulated conduit = L

Scale factor between two similar embankments = F

Definitions, Material Properties:

Total unit weight of embankment materials at conduit location = γ_T

Modulus of elasticity where equal for embankment and foundation, or where embankment modulus is not considered = E

Modulus of elasticity of embankment differing from that of the foundation = E_e

Modulus of elasticity of foundation = E_f

Poisson's ratio, taken equal for embankment and foundation = μ

Foundation shear strength = S

Maximum vertical applied pressure of embankment at conduit location = p

Components of Movement:

Maximum settlement of conduit beneath axis of the dam = δ

Maximum average vertical strain in compressible foundation beneath conduit = $\delta/d = \epsilon$

Horizontal opening or closing of the individual joints of the conduit = ΔL

Total horizontal displacement of the conduit outward from the center of movement =

Maximum horizontal strain of the conduit, taken at location of maximum ΔL = $\Delta L/L = \epsilon_h$

Ratio of total horizontal displacement to maximum settlement = L_h/δ

Ratio of maximum horizontal strain to maximum vertical strain = ϵ_h/ϵ_v

TABLE NO. 3, INFLUENCE OF VARYING PARAMETERS ON MOVEMENTS OF THE INTERFACE

Varying Parameter	Parameter Held Constant	Influence of Varying the Parameter on the Following:					
		Foundation Maximum Settlement δ	Foundation Maximum Vertical Strain δ/d	Interface Maximum Horizontal Displacement L_h	Interface Maximum Horizontal Strain ϵ_h	Displacement Ratio L_h/δ	Strain Ratio ϵ_h/ϵ_v (maximum)
Overall scale factor, F	b/d, b/h, E, γ_T, ν	increase with F^2	increase with F	increase with F^2	increase with F	constant	constant
Dam height, h	b/h, d, E, E, γ_T, ν	increase with h	increase with h	increase with h	approx. constant	constant	decrease with h
Dam base width, b	h, d, E, γ_T, ν	approx. constant	approx. constant	decrease with $\sim b$	decrease with $\sim b^2$	decrease with $\sim b$	decrease with $\sim b^2$
Foundation depth, d	h, b, E, γ_T, ν	increase with $\sim d$	approx. constant	increase with $\sim d^{2/3}$	increase with $\sim d^{2/3}$	decrease with $\sim d^{1/3}$	increase with $\sim d^{2/3}$
Embankment unit weight, γ_T	h, d, b, E, ν	increase with γ_T	increase with γ_T	increase with γ_T	increase with γ_T	constant	constant
Ratio E_e/F_f	E_e, h, d, b, γ_T, ν	increase with E_e/E_f	increase with E_e/E_f	increase with E_e/E_f	increase with E_e/E_f	approx. constant	approx. constant
Poisson's ratio, ν	h, d, b, E, γ_T	decrease with ν	decrease with ν	increase with $\sim\nu$	increase with $\sim\nu$	increase with $\sim\nu^2$	increase with $\sim\nu^2$

TABLE NO. 4-1, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

UPPER BLACK BEAR CREEK, NO. 71
NOBLE COUNTY, OKLAHOMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 340 feet. (b)
Length of dam measured along axis = 760 feet.
Length of dam within valley = 550 feet.
Embankment slopes, horizontal to vertical: (wide DS berm)
upstream = 3 to 1 ; downstream = 3 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 42 feet.
Maximum embankment height above invert of conduit = 42 feet. (h)
Thickness of fill below invert of conduit = 10 feet.
Thickness of compressible foundation soils = 22 feet.
Total thickness of compressible soils beneath conduit = 32 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 12 feet. (L)
Number of joints under observation = 30 .
Total length of conduit under observation = 360 feet.

SOIL DESCRIPTION

Fill: Reworked material from residual soils, CL, ML and SC. Sandstone and shale fragments from required excavation placed on slopes and berms.

Foundation Strata: (1) Alluvium, ML, N = 4 blows per foot, water level at ground surface. (2) Alluvium, CL with sandy seams. Underlain by shale and sandstone redbeds of Permian age.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	1008 to 966	14.5	--	--	0.5	110	126	24	70
(1)	966 to 945	22	23	18	0.6	110	129	17	63
(2)	945 to 934	23	27	17	0.6	106	126	25	80

TABLE NO. 4-2, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

POWDERMILL BROOK
HAMPDEN COUNTY, MASSACHUSETTS

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 390 feet. (b)
Length of dam measured along axis = 580 feet.
Length of dam within valley = 420 feet.
Embankment slopes, horizontal to vertical: 40' berms US and DS
upstream = 3-1/2 to 1 ; downstream = 3 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 47 feet.
Maximum embankment height above invert of conduit = 47 feet. (h)
Thickness of fill below invert of conduit = 5 feet.
Thickness of compressible foundation soils = 45 feet.
Total thickness of compressible soils beneath conduit = 50 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 48 inches. Nominal pipe length = 16 feet. (L)
Number of joints under observation = 25 .
Total length of conduit under observation = 400 feet.

SOIL DESCRIPTION

Fill: Recent river sand and silt, SM-SP, placed in berms and on outer slopes. Uppermost glacial lake silts and silty sands placed in center of dam. ML and SM.

Foundation Strata: (1)Recent river sand and gravel, SM and GW, loose, typical N=4. (2) Glacial lake deposit of varved silt and clay, sandy at top, generally ML-CL, with predominately clayey or sandy lenses, N increasing from 5 to 15 with depth. (3) Glacial till underlain by sandstone.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill (core)	205 to 157	20	Generally NP			100	120	4	90
(1)	157 to 147	---	Generally NP		--	--	--	--	--
(2)	147 to 107	32	27	20	0.9	94	124	6	95

TABLE NO. 4-3, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

MILL - PICAYUNE CREEK DAM
HARRISON COUNTY, IOWA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 220 feet. (b)
Length of dam measured along axis = 700 feet.
Length of dam within valley = 430 feet.
Embankment slopes, horizontal to vertical:
upstream = 3 to 1 ; downstream = 3 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 30 feet.
Maximum embankment height above invert of conduit = 24 feet. (h)
Thickness of fill below invert of conduit = 14 feet.
Thickness of compressible foundation soils = 42 feet.
Total thickness of compressible soils beneath conduit = 56 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 8 feet. (L)
Number of joints under observation = 22 .
Total length of conduit under observation = 168 feet.

SOIL DESCRIPTION

Fill: Generally loess from emergency spillway excavation, CL and ML. Maximum laboratory dry density = 99 pcf, optimum moisture = 21%.

Foundation Strata: (1) Wisconsin loess, relatively massive, yellow brown, small shells and carbonate concretions, CL and ML. N values 3 to 9 blows per foot. (2) Sappa formation, reworked loess, blue-gray, wood fragments and small shells, thin sand lenses, ML. N values 7 to 15 blows per foot. Underlain by Kansan till.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	1182 to 1142	24	38	23	0.7	96	119	27	90
(1)	1142 to 1119	31	35	24	0.8	94	123	24	90
(2)	1119 to 1103	33	36	26	0.8	91	121	21	92

TABLE NO. 4-4, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

HUFF CREEK NO. 2A
GREENVILLE COUNTY, SOUTH CAROLINA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 250 feet. (b)
Length of dam measured along axis = 760 feet.
Length of dam within valley = 330 feet.
Embankment slopes, horizontal to vertical: 20' and 30' DS and US berms
upstream = 2-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 33 feet.
Maximum embankment height above invert of conduit = 35 feet. (h)
Thickness of fill below invert of conduit = 5 feet.
Thickness of compressible foundation soils = 35 feet.
Total thickness of compressible soils beneath conduit = 40 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 16 feet. (L)
Number of joints under observation = 18 .
Total length of conduit under observation = 272 feet.

SOIL DESCRIPTION

Fill: Primarily residual soils weathered from schist obtained from emergency spillway excavation, ranging from MG to SM.

Foundation Strata: (1) Alluvium derived from residual soils, primarily SM. (2) Decomposed schist, grading downward to weathered parent rock, generally of low plasticity but comparatively high moisture content and low density. Highest water content at upper part of layer.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	783 to 748	20	Generally NP		0.6	98	118	18	40
(1)	748 to 729	30	Generally NP		0.8	90	117	4	30
(2)	729 to 710	45 to 30	36	33	0.9	88	119	5	33

TABLE NO. 4-5, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

BIG WEWOKA CREEK, NO.39

HUGHES COUNTY, OKLAHOMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 190 feet. (b)

Length of dam measured along axis = 2250 feet.

Length of dam within valley = 1500 feet.

Embankment slopes, horizontal to vertical:

upstream = 2-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 28 feet.

Maximum embankment height above invert of conduit = 30 feet. (h)

Thickness of fill below invert of conduit = 5 feet.

Thickness of compressible foundation soils = 35 feet.

Total thickness of compressible soils beneath conduit = 40 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 27 inches. Nominal pipe length = 12 feet. (L)

Number of joints under observation = 17 .

Total length of conduit under observation = 204 feet.

SOIL DESCRIPTION

Fill: Medium plastic CL and CL-ML materials, generally alluvial materials reworked from weathered soils residual from shales.

Foundation Strata: (1) Alluvial silt and clay, generally ML and CL. Water contents decrease with depth. N values in the range of 5 to 10. Underlain by shale of the Holdenville formation of Pennsylvanian Age.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	785 to 755	15	32	16	0.5	112	129	25	80
(1)	755 to 715	22	31	17	0.6	101	123	27	85

TABLE NO. 4-6, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

CYPRESS CREEK NO. 8

WEAKLEY COUNTY, TENNESSEE

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 140 feet. (b)

Length of dam measured along axis = 1400 feet.

Length of dam within valley = 1100 feet.

Embankment slopes, horizontal to vertical:

upstream = 3 to 1 ; downstream = 3 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 17 feet.

Maximum embankment height above invert of conduit = 21 feet. (h)

Thickness of fill below invert of conduit = 6 feet.

Thickness of compressible foundation soils = 13 feet.

Total thickness of compressible soils beneath conduit = 19 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 16 feet. (L)

Number of joints under observation = 11 .

Total length of conduit under observation = 176 feet.

SOIL DESCRIPTION

Fill: Silty alluvium and slope wash, primarily ML and SM-SC

Foundation Strata: (1) Alluvial clay, silt, sand formed by reworking underlying formation, primarily CL-ML in upper half grading to SC-SM in lower half. (2) Middle Eocene Gulf Coastal Plain Claiborne Group, clayey sands, dense, extends to about 150 foot depth, N values in excess of 50 blows per foot below El. 390.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	429 to 412	21	32	21	0.6	103	125	20	88
(1)	412 to 390	18	23	13	0.5	109	129	21	50

TABLE NO. 4-7, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS
UPPER WASHITA RIVER NO. 9
HEMPHILL COUNTY, TEXAS

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 260 feet. (b)
Length of dam measured along axis = 2700 feet.
Length of dam within valley = 1400 feet.
Embankment slopes, horizontal to vertical:
upstream = 2-1/2 to 1 ; downstream = 2-3/4 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 46 feet.
Maximum embankment height above invert of conduit = 43 feet. (h)
Thickness of fill below invert of conduit = 12 feet.
Thickness of compressible foundation soils = 22 feet.
Total thickness of compressible soils beneath conduit = 34 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 10 feet. (L)
Number of joints under observation = 28 .
Total length of conduit under observation = 280 feet.

SOIL DESCRIPTION

Fill: Generally silty sand, alluvial material reworked from Ogallala formation, SM and SM-SP.

Foundation Strata: (1) Alluvial sand, SM and SM-SP with clay pockets, N value roughly 15, reworked from Ogallala formation. (2) Alluvial clay, CL with sand lenses, reworked from Ogallala, N value roughly 3 to 5. (3) Tertiary Ogallala formation, dense sands to great depths, N value above 40.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	2569 to 2523	18	Generally NP		0.5	112	132	11	35
(1)	2523 to 2500	17	Generally NP		0.5	112	131	10	32
(2)	2500 to 2492	26	30	16	0.7	100	126	27	60

TABLE NO. 4-8, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

KICKAPOO SANDY CREEK NO. BJ-1

MURRAY COUNTY, OKLAHOMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 160 feet. (b)

Length of dam measured along axis = 1300 feet.

Length of dam within valley = 630 feet.

Embankment slopes, horizontal to vertical:

upstream = 2-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 26 feet.

Maximum embankment height above invert of conduit = 28 feet. (h)

Thickness of fill below invert of conduit = 6 feet.

Thickness of compressible foundation soils = 23 feet.

Total thickness of compressible soils beneath conduit = 29 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 12 feet. (L)

Number of joints under observation = 19 .

Total length of conduit under observation = 228 feet.

SOIL DESCRIPTION

Fill: Clayey and sandy alluvium reworked from soils weathered from shales and sandstone, generally with finer materials in core.

Foundation Strata: (1) Alluvial clay reworked from soils weathered from shale, medium to high plastic CL with lenses of ML and SM. Apparently fairly uniform, desiccated at surface, N values generally 20 to 10 (2) Underlain by shales and sandstone of Stratford formation of Permian age.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	891 to 864	13.5	Variable		0.5	111	126	22	60
(1)	864 to 835	22	42	21	0.6	105	128	32	76

TABLE NO. 4-9, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

QUARTERMASTER CREEK NO. 1
ROGER MILLS COUNTY, OKLAHOMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 200 feet. (b)
Length of dam measured along axis = 900 feet.
Length of dam within valley = 650 feet.
Embankment slopes, horizontal to vertical:
upstream = 2-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 29 feet.
Maximum embankment height above invert of conduit = 31 feet. (h)
Thickness of fill below invert of conduit = 19 feet.
Thickness of compressible foundation soils = 20 feet.
Total thickness of compressible soils beneath conduit = 39 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 27 inches. Nominal pipe length = 10 feet. (L)
Number of joints under observation = 23 .
Total length of conduit under observation = 230 feet.

SOIL DESCRIPTION

Fill: Clay and silt alluvium reworked from residual soils in emergency spillway excavation and borrow areas.

Foundation Strata: (1) Silty alluvium with sand lenses, ML, N values 10 to 15 in general. (2) Clayey alluvium reworked from residual soils weathered from underlying shales and siltstone, N values 4 to 10 generally minimum in 10' zone from Elev. 1857 to 1867, stiff and sandy in bottom 5 feet, underlain by Permian redbeds.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	1920 to 1890	14	--	--	0.4	116	132	--	--
(1)	1890 to 1870	22	21	19	0.5	101	123	12	54
(2)	1870 to 1850	26	26	19	0.6	97	122	23	65

TABLE NO. 4-10, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

TOBESOFKEE CREEK NO. 70

LAMAR COUNTY, GEORGIA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 300 feet. (b)

Length of dam measured along axis = 1200 feet.

Length of dam within valley = 400 feet.

Embankment slopes, horizontal to vertical:

upstream = 2-3/4 to 1 ; downstream = 3 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 41 feet.

Maximum embankment height above invert of conduit = 42 feet. (h)

Thickness of fill below invert of conduit = 7 feet.

Thickness of compressible foundation soils = 8 feet.

Total thickness of compressible soils beneath conduit = 15 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 42 inches. Nominal pipe length = 16 feet. (L)

Number of joints under observation = 21 .

Total length of conduit under observation = 313 feet.

SOIL DESCRIPTION

Fill: Micaceous silt and sand obtained primarily from required excavation of emergency spillways, ML, SM, SC.

Foundation Strata: (1) Micaceous silty and clayey sand, apparently partially reworked, derived from weathering of underlying schist or gneiss, SM and SM-SC, N values generally in range of 10 to 25 increasing with depth. (2) Underlain by decomposed schist, N values generally above 30 or 50, increasing proportion of rock lenses with depth. Grades into parent crystalline rocks of Piedmont.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	644 to 602	23	35 (and	19 NP)	0.6	102	128	25	50
(1)	602 to 587	24	32 (and	19 NP)	0.6	103	128	18	40

TABLE NO. 4-11, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

DAM NO. 6, WHITE CLAY, BREWERY, WHISKEY CREEKS
ATCHISON COUNTY, KANSAS

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 200 feet. (b)
Length of dam measured along axis = 650 feet.
Length of dam within valley = 300 feet.
Embankment slopes, horizontal to vertical:
upstream = 3 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 28 feet.
Maximum embankment height above invert of conduit = 35 feet. (h)
Thickness of fill below invert of conduit = 5 feet.
Thickness of compressible foundation soils = 5 feet.
Total thickness of compressible soils beneath conduit = 10 feet. (d)

CONDUIT CHARACTERISTICS (concrete box of square cross section)

Nominal diameter = 54 inches. Nominal pipe length = 20 feet. (L)
Number of joints under observation = 9 .
Total length of conduit under observation = 193 feet.

SOIL DESCRIPTION

Fill: Probably generally Kansan till, CL.

Foundation Strata: (1) Reworked Loveland loess in valley bottom, CL with sand lenses. (2) Loess is underlain by Kansan till, clayey sand, yellow, moist, 5% gravel, SC, SM&CL. Kansan till underlain by Atchison sand and Nebraskan till.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	923 to 894	17	--	--	0.5	109	128	--	--
(1)	894 to 884	17	29	15	0.5	96	112	23	78
(2)	Below 884	16	27	15	0.4	113	131	21	50

TABLE NO. 4-12, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

UPPER WASHITA RIVER, NO. 57
ROGER MILLS COUNTY, OKLAHOMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 370 feet. (b)
Length of dam measured along axis = 1200 feet.
Length of dam within valley = 800 feet.
Embankment slopes, horizontal to vertical:
upstream = 2-1/2 to 1, 4 to 1 ; downstream = 2 to 1, 3 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 50 feet.
Maximum embankment height above invert of conduit = 53 feet. (h)
Thickness of fill below invert of conduit = 5 feet.
Thickness of compressible foundation soils = 15 feet.
Total thickness of compressible soils beneath conduit = 20 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 36 inches. Nominal pipe length = 12 feet. (L)
Number of joints under observation = 39 .
Total length of conduit under observation = 468 feet.

SOIL DESCRIPTION

Fill: Alluvial sand, silt and clay derived from weathered shale placed in core, SM, ML, CL, 95% of laboratory maximum density. Shale fragments from required excavation placed on outer slopes.

Foundation Strata: (1) Silty alluvium derived from weathered shale, ML, N values 4 to 8. (2) Sandy alluvium, SP and SM, N values 6 to 14. (3) Clayey alluvium derived from weathered shale without extensive reworking, CL, N values 4 to 8. (4) Permian Quartermaster Formation, Doxey Red Shale member.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	2177 to 2127	16	varies	varies	0.4	110	128	varies	
(1)	2127 to 2119	25	33	23	0.6	96	120	15	60
(2)	2119 to 2110	16	18	15	0.5	108	125	10	40
(3)	2110 to 2106	28	38	24	0.7	98	125	30	90

TABLE NO. 4-13, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

UPPER RED ROCK CREEK NO. 33
GARFIELD COUNTY, OKLAHOMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 190 feet. (b)
Length of dam measured along axis = 1250 feet.
Length of dam within valley = 350 feet.
Embankment slopes, horizontal to vertical:
upstream = 2-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 26 feet.
Maximum embankment height above invert of conduit = 27 feet. (h)
Thickness of fill below invert of conduit = 7 feet.
Thickness of compressible foundation soils = 12 feet.
Total thickness of compressible soils beneath conduit = 19 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 12 feet. (L)
Number of joints under observation = 18.
Total length of conduit under observation = 204 feet.

SOIL DESCRIPTION

Fill: Reworked material from residual soils derived from shales primarily CL. Finest material placed in conduit trench and central portion of embankment.

Foundation Strata: (1) Alluvial clays, CL, derived from residual soils weathered from underlying shales. Apparently somewhat stiffened at the surface. N values range from 10 to 20 in upper half of layer and from 5 to 10 in lower half. Underlain by shales of Garber formation of Permian age.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	1086 to 1058	14	--	--	0.5	112	128	--	--
(1)	1058 to 1039	21	37	19	0.6	103	125	32	75

TABLE NO. 4-14, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

BRISTOWS CREEK NO. 1
ETOWAH COUNTY, ALABAMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 200 feet. (b)
Length of dam measured along axis = 700 feet.
Length of dam within valley = 450 feet.
Embankment slopes, horizontal to vertical:
upstream = 3 to 1 ; downstream = 3 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 26 feet.
Maximum embankment height above invert of conduit = 28 feet. (h)
Thickness of fill below invert of conduit = 5 feet.
Thickness of compressible foundation soils = 24 feet.
Total thickness of compressible soils beneath conduit = 29 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 16 feet. (L)
Number of joints under observation = 16 .
Total length of conduit under observation = 227 feet.

SOIL DESCRIPTION

Fill: Residual soil derived from shale and sandstone of Silurian Red Mountain formation. Borrow materials well mixed because of vertical bedding in the borrow area.

Foundation Strata: (1) Colluvial and residual cherty clays and clayey sands derived primarily from weathering in place of Mississippian Age limestones. Becoming more sandy and probably much less compressible with depth. (2) Underlain by weathered and jointed limestone with pockets of residual soil consisting of clayey chert sands. (3) Parent limestone, moderately jointed.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	915 to 888	18	--	--	0.5	111	131	--	--
(1)	888 to 859	22	40	19	0.6	112	137	24	53

TABLE NO. 4-15, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

PORTERS CREEK NO. 11
TIPPAH COUNTY, MISSISSIPPI

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 150 feet. (b)
Length of dam measured along axis = 1000 feet.
Length of dam within valley = 500 feet.
Embankment slopes, horizontal to vertical:
upstream = 3 to 1; downstream = 3 to 1.

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 21 feet.
Maximum embankment height above invert of conduit = 22 feet. (h)
Thickness of fill below invert of conduit = 5 feet.
Thickness of compressible foundation soils = 23 feet.
Total thickness of compressible soils beneath conduit = 28 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 16 feet. (L)
Number of joints under observation = 14.
Total length of conduit under observation = 200 feet.

SOIL DESCRIPTION

Fill: Material from emergency spillway required excavation, CL-ML, CL and SC-SM.

Foundation Strata: (1) Alluvial-colluvial clay with sand lenses, derived from underlying Tertiary strata, N values generally 5 to 15 increasing with depth, water contents near plastic limit. (2) Porters Creek formation of Paleocene age, MH or shale, N values 35 to 50, gray color, finely laminated.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	532 to 510	13	--	--	0.4	120	135	--	--
(1)	510 to 483	20	32	18	0.5	105	126	25	60

TABLE NO. 4-16, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

TEWAUKON NO. T1-A
SARGENT COUNTY, NORTH DAKOTA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 245 feet. (b)
Length of dam measured along axis = 3200 feet.
Length of dam within valley = 800 feet.
Embankment slopes, horizontal to vertical:
upstream = 3 to 1; downstream = 3 to 1.

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 34 feet.
Maximum embankment height above invert of conduit = 35 feet. (h)
Thickness of fill below invert of conduit = 10 feet.
Thickness of compressible foundation soils = 14 feet.
Total thickness of compressible soils beneath conduit = 24 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 16 feet. (L)
Number of joints under observation = 15.
Total length of conduit under observation = 230 feet.

SOIL DESCRIPTION

Fill: Reworked clayey till or weathered clayey till from abutments and required excavation, generally CL.

Foundation Strata: (1) Alluvial-colluvial silt and clay, sandy at top derived from till, generally CL and ML, N values generally below 10, becoming denser with depth (2) Weathered till clayey, CL, N values 10 to 30 becoming denser with depth. (3) Unweathered till, N values 40 to 65, underlain by stiff varved lake sediments and Pierre shale.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	1352 to 1318	16	--	--	0.5	108	125	--	--
(1)	1318 to 1302	27	39	26	0.7	94	119	16	74
(2)	1302 to 1294	18	31	16	0.5	109	129	24	70

TABLE NO. 4-17, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

TEWAUKON NO. T-2
SARGENT COUNTY, NORTH DAKOTA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 200 feet. (b)
Length of dam measured along axis = 1700 feet.
Length of dam within valley = 1000 feet.
Embankment slopes, horizontal to vertical:
upstream = 3-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 29 feet.
Maximum embankment height above invert of conduit = 32 feet. (h)
Thickness of fill below invert of conduit = 5 feet.
Thickness of compressible foundation soils = 10 feet.
Total thickness of compressible soils beneath conduit = 15 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 16 feet. (L)
Number of joints under observation = 13 .
Total length of conduit under observation = 208 feet.

SOIL DESCRIPTION

Fill: Alluvium and colluvium derived from glacial till, generally CL.

Foundation Strata: (1) Alluvium and colluvium from glacial till, CL, N values generally 5 to 10. (2) Weathered glacial till, CL, N values 10 to 15. (3) Unweathered till CL, N values 8 to 15, underlain by older glacial till.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	1269 to 1240	21	--	--	0.6	102	123	--	--
(1)	1240 to 1228	27	43	23	0.7	97	123	25	68
(2)	1220 to 1228								
(3)	1228 to 1188	26	40	22	0.7	98	123	21	66

TABLE NO. 4-18, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

UPPER WABASH NO. 1
DARKE COUNTY, OHIO

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 170 feet. (b)
Length of dam measured along axis = 680 feet.
Length of dam within valley = 400 feet.
Embankment slopes, horizontal to vertical:
upstream = 3 to 1 ; downstream = 2-1/2 to 1.

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 30 feet.
Maximum embankment height above invert of conduit = 32 feet. (h)
Thickness of fill below invert of conduit = 0 feet.
Thickness of compressible foundation soils = 26 feet.
Total thickness of compressible soils beneath conduit = 26 feet. (d)

CONDUIT CHARACTERISTICS (Cast-in-place, square cross section)

Nominal diameter = 48 inches. Nominal pipe length = 34 feet. (L)
Number of joints under observation = 5 .
Total length of conduit under observation = 178 feet.

SOIL DESCRIPTION

Fill: Clayey upland till similar to foundation till, generally CL.

Foundation Strata: (1) Alluvium derived from glacial till, CL, N values 15 to 20. (2) Glacial till, clayey, CL, N values 15 to 40, generally increasing with depth. Underlain by dolomitic limestone of Guelph formation of Silurian age.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	1024 to 994	18	--	--	0.5	107	126	--	--
(1)	994 to 988	13	34	20	0.4	112	127	23	71
(2)	988 to 966	13	24	15	0.4	114	129	23	68

TABLE NO. 4-19, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

SUGAR CREEK NO. 41
CADDO COUNTY, OKLAHOMA

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 248 feet. (b)
Length of dam measured along axis = 750 feet.
Length of dam within valley = 450 feet.
Embankment slopes, horizontal to vertical:
upstream = 3-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 38 feet.
Maximum embankment height above invert of conduit = 39 feet. (h)
Thickness of fill below invert of conduit = 7 feet.
Thickness of compressible foundation soils = 41 feet.
Total thickness of compressible soils beneath conduit = 48 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 12 feet. (L)
Number of joints under observation = 25 .
Total length of conduit under observation = 288 feet.

SOIL DESCRIPTION

Fill: Primarily silty sand with some sandstone fragments derived from weathering of parent rocks, SM.

Foundation Strata: (1) Loose to medium compact orange silty medium sand, SM with occasional ML lenses, N values generally 10 to 20 with some loose lenses with N less than 5. (2) Compact silty medium sand, SM, N values 20 to 30. Both layers derived from underlying Rock Springs and Marlow sandstones of Permian age.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer 0.002 mm	Grain Size per cent finer 200 sieve
Fill	1430 to 1391	11	Non	plastic	0.5	109	121	--	--
(1)	1391 to 1343	15	Non	plastic	0.6	106	122	8	44

TABLE NO. 4-20, SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS

KICKAPOO NO. 4
COKE COUNTY, TEXAS

GENERAL EMBANKMENT CHARACTERISTICS

Width of dam at base of maximum section = 185 feet. (b)
Length of dam measured along axis = 2400 feet.
Length of dam within valley = 1200 feet.
Embankment slopes, horizontal to vertical:
upstream = 2-1/2 to 1 ; downstream = 2-1/2 to 1 .

CONDITIONS AT CONDUIT LOCATION

Maximum embankment height above original ground = 28 feet.
Maximum embankment height above invert of conduit = 31 feet. (h)
Thickness of fill below invert of conduit = 6 feet.
Thickness of compressible foundation soils = 5 feet.
Total thickness of compressible soils beneath conduit = 11 feet. (d)

CONDUIT CHARACTERISTICS

Nominal diameter = 30 inches. Nominal pipe length = 10 feet. (L)
Number of joints under observation = 21 .
Total length of conduit under observation = 200 feet.

SOIL DESCRIPTION

Fill: Sandy clay obtained from residual soils derived from parent shale, CL.

Foundation Strata: (1) Alluvial clay, CL, sandy derived from weathering of underlying shale. Water contents below plastic limit. Underlain by shale and sandstone of San Angelo formation of Permian age.

SOIL IDENTIFICATION PROPERTIES

Strata	Elevs.	Natural Water Content %	Liquid Limit	Plastic Limit	Void Ratio	Dry Density pcf	Total Density pcf	Grain Size per cent finer	
								0.002 mm	200 sieve
Fill	2002 to 1974	13	32	16	0.3	114	129	33	80
(1)	1974 to 1960	13	30	16	0.3	112	127	30	65

TABLE NO. 5, SUMMARY OF OBSERVATION DATA

Case number	1	2	3	4
Structure	Upper Black Bear Creek No. 71	Powdermill Brook	Mill-Picayune Creek	Huff Creek No. 2A
Sequence of dates				
Construction starts:	March 26, 1962	July 1963	May 13, 1963	May, 1962
Construction ends:	July 11, 1962	Oct. 28, 1963	July 5, 1963	Aug. 17, 1962
Original survey:	June 18, 1962	July 16, 1963	June 17, 1963	June 19, 1962
Latest survey:	Nov. 17, 1965	Nov. 1, 1965	July 14, 1967	Aug. 9, 1963
Time interval, months				
Original survey to end construction:	0.7	3.4	0.6	2.1
End construction to latest survey:	40.3	24.1	48.3	11.6
Total observed time:	41.0	27.6	48.9	13.7
Number of surveys	14	10	11	7
Maximum settlement, feet				
End construction: (δ)	1.15	0.58	1.24	1.41
Latest survey: (δ)	1.31	0.67	1.71	1.59
Maximum joint opening, inches (ΔL)				
End construction:	0.97	0.45	0.86	0.25
Latest survey:	1.03	0.52	0.91	0.30
Maximum joint rotation, radians:	0.014	0.0044	0.023	0.013
Total extension, inches				
End construction:	3.3	2.0	3.4	0.81
Latest survey:	3.8	2.3	3.8	1.25
Length of tension zone, feet:	60	110	56	90
Special observed conditions:	Cracking at 7 joints, probably due to rotation. Vertical offset 1" at Jt. 10	Joint patching materials softened by flowing water	No pipe cracking or distress observed	No pipe cracking or distress observed

TABLE NO. 5, SUMMARY OF OBSERVATION DATA (Cont'd)

Case number	5	6	7	8
Structure	Big Wewoka Creek No. 39	Cypress Creek No. 8	Upper Washita River No. 9	Kickapoo-Sandy Creek No. BJ-1
Sequence of dates Construction starts: Construction ends: Original survey: Latest survey:	 Sept. 1960 Dec. 7, 1960 Oct. 24, 1960 Nov. 4, 1965	 July 1963 Aug. 28, 1963 July 23, 1963 June 20, 1966	 June 1962 Nov. 30, 1962 June 8, 1962 Dec. 17, 1963	 June 1963 Aug. 12, 1963 July 19, 1963 Nov. 30, 1966
Time interval, months Original survey to end construction: End construction to latest survey: Total observed time:	 1.5 58.9 60.4	 1.1 33.8 34.9	 5.7 12.6 18.3	 0.7 39.6 40.3
Number of surveys	9	10	9	12
Maximum settlement, feet End construction: (δ) Latest survey: (δ)	 0.40 0.42	 0.14 0.17	 0.33 0.36	 0.19 0.28
Maximum joint opening, inches (ΔL) End construction: Latest survey:	 0.44 0.42	 0.19 0.28	 0.21 0.31	 0.16 0.50
Maximum joint rotation, radians:	0.011	0.0044	0.0050	0.0050
Total extension, inches End construction: Latest survey: Length of tension zone, feet:	 1.3 2.3 108	 0.35 0.85 64	 0.70 1.50 80	 almost 0 6.5 full 160 ft.
Special observed conditions:	Cracking at joints 7 & 12, probably due to rotation	No pipe cracking or distress observed	No pipe cracking or distress observed	No pipe cracking. Joint 1 opened 1/2" suddenly

TABLE NO. 5, SUMMARY OF OBSERVATION DATA (Cont'd)

Case number	9	10	11	12
Structure	Quartermaster Creek No. 1	Tobesofkee Creek No. 70	White Clay-Brewery Creek No. 6	Upper Washita River No. 57
Sequence of dates				
Construction starts:	Sept. 1961	Oct. 1962	May 17, 1963	Sept. 1960
Construction ends:	Feb. 1962	April 26, 1963	July 11, 1963	Jan. 30, 1961
Original survey:	Jan. 30, 1962	Dec. 18, 1962	May 17, 1963	Nov. 15, 1960
Latest survey:	June 1, 1965	May 8, 1964	Aug. 10, 1964	July 17, 1964
Time interval, months				
Original survey to end construction:	1.7	4.3	1.7	2.5
End construction to latest survey:	38.3	12.3	13.0	41.5
Total observed time:	40.0	16.6	14.7	44.0
Number of surveys	10	9	11	14
Maximum settlement, feet				
End construction: (δ)	1.07	0.38	0.13	0.66
Latest survey: (δ)	1.22	0.49	0.145	0.96
Maximum joint opening, inches (ΔL)				
End construction:	0.50	0.19	0.22	0.43
Latest survey:	0.56	0.25	0.31	0.72
Maximum joint rotation, radians:	0.014	0.0070	0.0040	0.016
Total extension, inches				
End construction:	2.4	0.33	0.64	1.15
Latest survey:	3.0	0.47 (Total net=0)	1.11	2.97
Length of tension zone, feet:	70	64	80	120
Special observed conditions:	Hairline crack at Jt. 1 only.	Radial cracks at Jts. 10, 12 and 13	Joints 8, 9 in compression on all faces.	Longitudinal cracking in 9 pipe sections Several circular cracks.

TABLE NO. 5, SUMMARY OF OBSERVATION DATA (Cont'd)

Case number	13	14	15	16
Structure	Upper Red Rock Creek No. 33	Bristows Creek No. 1	Porters Creek No. 11	Tewaukon No. T1-A
Sequence of dates				
Construction starts:	Aug. 1963	Sept. 1962	May 1963	Aug. 1962
Construction ends:	Feb. 11, 1964	June 13, 1963	Aug. 21, 1963	Oct. 7, 1963
Original survey:	Jan. 3, 1964	Nov. 5, 1962	May 15, 1963	June 1963
Latest survey:	Oct. 28, 1966	June 22, 1965	Nov. 10, 1965	Nov. 22, 1965
Time interval, months				
Original survey to end construction:	1.4	7.3	3.2	4.2
End construction to latest survey:	32.6	24.3	26.6	24.5
Total observed time:	34.0	31.6	29.8	28.7
Number of surveys	9	11	8	7
Maximum settlement, feet				
End construction: (δ)	0.19	0.40	0.08	0.28
Latest survey: (δ)	0.26	0.59	0.135	0.32
Maximum joint opening, inches (ΔL)				
End construction:	0.19	0.17	0.03	0.19
Latest survey:	0.34	0.24	0.10	0.28
Maximum joint rotation, radians:	0.010	0.009	0.003	0.003
Total extension, inches				
End construction:	0.6	0.33	0.10	0.85
Latest survey:	1.5	0.36	0.45	1.3
Length of tension zone, feet:	120	48 (DS of axis)	96	110
Special observed conditions:	No pipe cracking or distress noted	Slight seepage at 3 joints during high pool	No pipe cracking or distress noted	No pipe cracking or distress noted

TABLE NO. 5, SUMMARY OF OBSERVATION DATA (Cont'd)

Case number	17	18	19	20
Structure	Tewaukon No. T-2	Upper Wabash No. 1	Sugar Creek No. 41	Kickapoo No. 4
Sequence of dates				
Construction starts:	July 1965	Sept. 1963	Jan. 1963	May 1962
Construction ends:	Oct. 25, 1965	Dec. 23, 1963	March 26, 1963	Sept. 6, 1962
Original survey:	Sept. 7, 1965	Oct. 18, 1963	Feb. 15, 1963	June 19, 1962
Latest survey:	June 16, 1966	Feb. 10, 1967	May 4, 1966	Sept. 17, 1963
Time interval, months				
Original survey to end construction:	1.6	2.2	1.3	2.5
End construction to latest survey:	7.7	37.5	37.3	12.4
Total observed time:	9.3	39.7	38.6	14.9
Number of surveys	6	10	11	8
Maximum settlement, feet				
End construction: (δ)	0.48	0.09	0.40	0.11
Latest survey: (δ)	0.54	0.09	0.79	0.14
Maximum joint opening, inches (ΔL)				
End construction:	0.35	0.25	0.16	0.031
Latest survey:	0.47	0.44	0.35	0.062
Maximum joint rotation, radians:	0.015	0.002	0.020	0.009
Total extension, inches				
End construction:	1.25	0.75	1.5	0.45
Latest survey:	1.35	1.0	3.3	0.65
Length of tension zone, feet:	80	102	108	90
Special observed conditions:	No pipe cracking or distress observed	No pipe cracking or distress observed	No pipe cracking or distress observed	No pipe cracking or distress noted

TABLE NO. 6, SUMMARY OF INTERPRETED DATA

Case number	1	2	3	4
Structure	Upper Black Bear Creek No. 71	Powdermill Brook	Mill-Picayune Creek	Huff Creek No. 2A
Equivalent dam width (b), feet:	340	390	220	250
Equivalent dam height (h), feet:	42	47	30	35
Foundation depth (d), feet:	32	50	50	40
Height ratio, b/h :	8.1	8.3	7.3	7.1
Depth ratio, b/d :	10.6	7.8	4.4	6.2
Maximum vertical strain, δ/d :	0.041	0.013	0.034	0.040
Maximum horizontal strain $\Delta L/L$:	0.0077	0.0027	0.0095	0.0016
Observed strain ratio, ϵ_h/ϵ_v	0.19	0.20	0.28	0.04
Theoretical strain ratio	0.16	0.20	0.35	0.27
Maximum vertical pressure, p, ksf	5.3	5.6	3.6	4.3
Estimated foundation shear strength, ksf	1.2	1.6	1.5	1.9
Elastic limit of vertical pressure: $p = \frac{s}{2} \cdot \frac{b}{d}$ ksf	5.6	6.2	3.3	5.9

TABLE NO. 6. SUMMARY OF INTERPRETED DATA (Cont'd)

Case number	5	6	7	8
Structure	Big Wewoka Creek No. 39	Cypress Creek No. 8	Upper Washita River No. 9	Kickapoo-Sandy Creek No. BJ-1
Equivalent dam width (b), feet:	190	140	260	160
Equivalent dam height (h), feet:	28	20	43	27
Foundation depth (d), feet:	40	19	34	29
Height ratio, b/h :	6.8	7.0	6.1	5.9
Depth ratio, b/d :	4.8	7.4	7.6	5.5
Maximum vertical strain, δ/d :	0.010	0.0089	0.011	0.0096
Maximum horizontal strain $\Delta L/L$:	0.0031	0.0015	0.0026	0.0035
Observed strain ratio, ϵ_h/ϵ_v	0.31	0.16	0.24	0.36
Theoretical strain ratio	0.35	0.24	0.26	0.37
Maximum vertical pressure, p, ksf	3.6	2.5	5.7	3.4
Estimated foundation shear strength, ksf	1.4	1.2	1.7	1.3
Elastic limit of vertical pressure: $p = \frac{s}{2} \cdot \frac{b}{d}$ ksf	3.4	4.4	6.5	3.6

TABLE NO. 6, SUMMARY OF INTERPRETED DATA (Cont'd)

Case number	9	10	11	12
Structure	Quarter-master Creek No. 1	Tobesofkee Creek No. 70	White Clay-Brewery Creek No. 6	Upper Washita River No. 57
Equivalent dam width (b), feet:	200	300	200	340
Equivalent dam height (h), feet:	30	42	30	51
Foundation depth (d), feet:	39	15	10	20
Height ratio, b/h :	6.7	7.2	6.7	6.7
Depth ratio, b/d :	5.1	20	20	17
Maximum vertical strain, δ/d :	0.029	0.033	0.015	0.048
Maximum horizontal strain $\Delta L/L$:	0.0047	0.0013	0.0013	0.0037
Observed strain ratio, ϵ_h/ϵ_v	0.16	0.04	0.09	0.08
Theoretical strain ratio	0.32	0.12	0.13	0.14
Maximum vertical pressure, p, ksf	4.0	5.4	3.8	6.4
Estimated foundation shear strength, ksf	1.8	2.0	1.5	2.6
Elastic limit of vertical pressure: $p = \frac{s}{2} \cdot \frac{b}{d}$ ksf	4.6	20	15	22

TABLE NO. 6, SUMMARY OF INTERPRETED DATA (Cont'd)

Case number	13	14	15	16
Structure	Upper Red Rock Creek No. 33	Bristows Creek No. 1	Porters Creek No. 11	Tewaukon No. T1-A
Equivalent dam width (b), feet:	165	200	150	245
Equivalent dam height (h), feet:	27	27	22	35
Foundation depth (d), feet:	19	29	28	24
Height ratio, b/h :	6.1	7.4	6.8	7.0
Depth ratio, b/d :	8.7	6.9	5.4	10.2
Maximum vertical strain, δ/d :	0.014	0.020	0.005	0.013
Maximum horizontal strain $\Delta L/L$:	0.0024	0.0013	0.0005	0.0015
Observed strain ratio, ϵ_h/ϵ_v	0.17	0.06	0.10	0.11
Theoretical strain ratio	0.24	0.24	0.30	0.19
Maximum vertical pressure, p, ksf	3.5	3.5	3.0	4.4
Estimated foundation shear strength, ksf	1.0	1.9	1.7	1.2
Elastic limit of vertical pressure: $p = \frac{s}{2} \cdot \frac{b}{d}$ ksf	4.3	5.9	4.6	6.1

TABLE NO. 6, SUMMARY OF INTERPRETED DATA (Cont'd)

Case number	17	18	19	20
Structure	Tewaukon No. T-2	Upper Wabash No. 1	Sugar Creek No. 41	Kickapoo No. 4
Equivalent dam width (b), feet:	200	170	248	185
Equivalent dam height (h), feet:	29	30	39	30
Foundation depth (d), feet:	45	26	48	11
Height ratio, b/h :	6.9	5.7	6.4	6.2
Depth ratio, b/d :	4.5	6.5	5.2	16.8
Maximum vertical strain, δ/d :	0.012	0.0038	0.016	0.013
Maximum horizontal strain $\Delta L/L$:	0.0025	0.0011	0.0024	0.00052
Observed strain ratio, ϵ_h/ϵ_v	0.21	0.28	0.15	0.04
Theoretical strain ratio	0.35	0.33	0.33	0.15
Maximum vertical pressure, p, ksf	3.6	3.8	4.7	3.9
Estimated foundation shear strength, ksf	1.7	2.5	2.4	2.0
Elastic limit of vertical pressure: $p = \frac{s}{2} \cdot \frac{b}{d}$ ksf	3.8	8.1	6.2	16.8

TABLE NO. 7, SUMMARY OF DATA FOR EMBANKMENTS UNDER OBSERVATION

Factor	One case smaller than:	Five cases smaller than:	Ten cases smaller, Ten cases larger:	Five cases larger than:	One case larger than:
Dam height, h, ft.	22	28	30	35	47
Dam base, b, ft.	150	190	200	250	340
Foundation depth, d, ft.	11	20	29	40	50
Ratio b/d	4.5	5.4	7.5	10.2	20
Ratio b/h	5.9	6.4	6.8	7.1	8.1
Max. Settlement δ, ft.	0.14	0.26	0.45	0.79	1.59
Horiz. Ext. L_h, in.	0.23	0.50	0.70	1.5	1.9
Ratio L_h / δ	0.04	0.10	0.16	0.21	0.46
Max. Joint Opening, ΔL, in.	0.10	0.28	0.35	0.50	0.91
Av. Settlement Slope $2\delta/b$, radians	0.0014	0.0026	0.0034	0.0059	0.0127
Max. Joint Rotation, radians	0.003	0.004	0.009	0.014	0.020
Ratio: $\frac{\text{Rotation}}{2\delta/b}$	1.1	1.5	1.8	2.8	3.2

TABLE NO. 7, SUMMARY OF DATA FOR EMBANKMENTS UNDER OBSERVATION
(Continued)

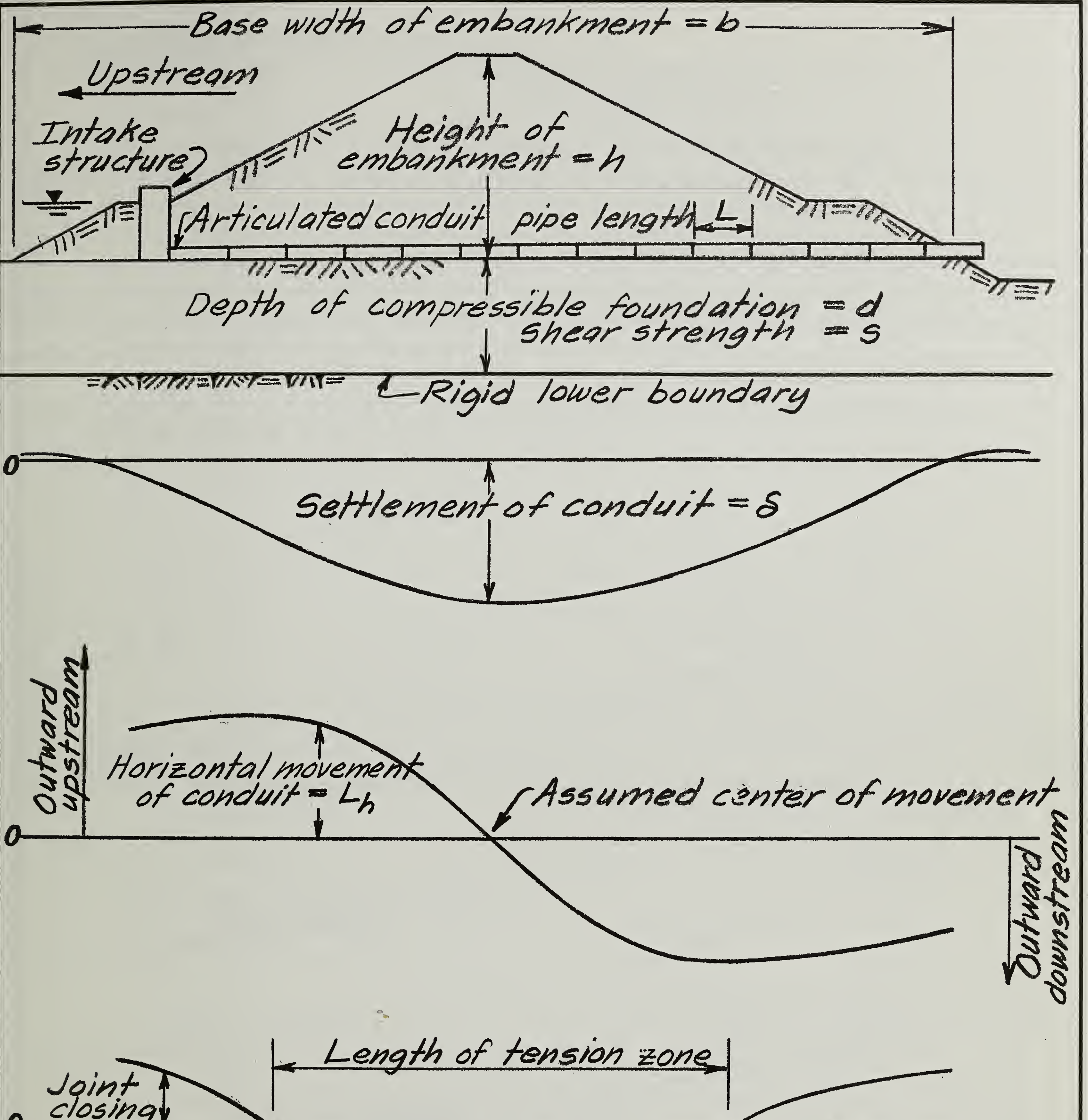
Factor	One case smaller than:	Five cases smaller than:	Ten cases smaller, Ten cases larger:	Five cases larger than:	One case larger than:
Vertical strain, ϵ_v	0.005	0.011	0.013	0.029	0.041
Horizontal strain ϵ_h	0.00052	0.0013	0.0024	0.0031	0.0077
Observed ratio ϵ_h/ϵ_v	0.04	0.09	0.16	0.21	0.31
Theoretical ratio ϵ_h/ϵ_v	0.13	0.19	0.24	0.33	0.35
Ratio: $\frac{\text{Obs. } \epsilon_h/\epsilon_v}{\text{Theo. } \epsilon_h/\epsilon_v}$	0.25	0.45	0.63	0.85	1.00
Maximum vertical pressure, p, ksf	3.0	3.6	3.9	4.7	5.7
Stress ratio= 2 pd/sb	0.25	0.57	0.75	0.90	1.06
Observed E= p/ϵ_v	110	160	260	350	600
% Settlement at end construction	59	73	85	89	95
%Joint Opening at end construction	32	57	70	84	95
Time interval for construction, months	0.7	1.4	1.9	3.2	4.3
Reading period after construction, months	12	13	30	38	48

TABLE NO. 8, COMPARISON OF OBSERVED AND THEORETICAL STRAIN RATIOS

Case No.	Geologic Category	Foundation Properties WC	Foundation Properties LL	b/d Ratio	Observed $E = P/\epsilon_v$	Stress Ratio $\frac{2pd}{sb}$	Strain Ratio ϵ_h/ϵ_v Observed	Strain Ratio ϵ_h/ϵ_v Theoretical	Strain Ratio ϵ_h/ϵ_v Observed/Theoretical
		%			ksf				
1	A	23	25	10.6	130	.95	.19	.16	1.19
5		22	31	4.8	360	1.06	.31	.35	.89
8		22	42	5.5	350	.95	.36	.37	.97
9		24	24	5.1	140	.87	.16	.32	.50
12		22	Varies	17	130	.29	.08	.14	.57
13		21	37	8.7	250	.81	.17	.24	.71
14		22	40	6.9	180	.59	.06	.24	.25
19		15	NP	5.2	200	.76	.16	.24	.45
20		13	30	16.8	300	.23	.04	.15	.27
6	B	18	23	7.4	280	.57	.16	.24	.67
7		26	30	7.6	520	.88	.24	.26	.92
15		20	32	5.4	600	.65	.10	.30	.33
4	C	36	36	6.2	110	.73	.04	.27	.15
10		24	32	20	160	.27	.04	.12	.33
2	D	32	27	7.8	480	.90	.20	.20	1.00
3		32	35	4.4	110	1.09	.28	.35	.80
11		17	29	20	180	.25	.09	.13	.69
16		27	39	10.2	340	.66	.11	.19	.58
17		27	42	4.5	300	.95	.21	.35	.60
18		13	24	6.5	1000	.48	.28	.33	.85

TABLE 9, SUMMARY OF DATA FROM REFERENCE 4

Case No.	Height feet	Ratio b/d	Ratio b/h	Vertical Strain ϵ_v	Adjusted Horiz. Strain ϵ_h	Observed Strain Ratio ϵ_h/ϵ_v	Theoretical ϵ_h/ϵ_v Plate 12	Ratio Observed to Theoretical ϵ_h/ϵ_v	Total Horiz. Ext. $2L_h$ feet	Settlement δ feet	Ratio L_h/δ
1	40	10.0	7.5	.045	.012	.27	.18	1.50	.385	1.35	.14
2	48	8.0	8.3	.014	.0035	.26	.20	1.30	.130	0.68	.10
3	30	4.4	7.3	.034	.010	.31	.35	0.89	.251	1.70	.07
4	33	6.1	7.4	.040	.0021	.05	.26	0.19	.156	1.59	.05
5	27	4.8	7.0	.011	.0035	.32	.34	0.94	.247	0.43	.33
6	20	4.2	7.0	.005	.0018	.35	.37	0.95	.089	0.17	.26
7	45	6.7	5.8	.009	.0024	.27	.31	0.87	.067	0.35	.10
8	25	5.5	6.4	.009	.0042	.45	.32	1.41	.520	0.28	.93
9	29	4.9	7.3	.029	.0051	.17	.30	0.57	.135	1.25	.05
10	40	12.0	7.5	.018	.0022	.12	.16	0.75	.026	0.40	.03
11	30	10.5	6.3	.009	.0013	.15	.21	0.71	.078	0.16	.25
12	50	16.0	7.0	.046	.0035	.08	.14	0.57	.151	0.92	.08
13	28	9.0	5.8	.014	.0019	.12	.25	0.48	.120	0.28	.21
Median	30	6.7	7.0	.014	.0035	.26	.26	0.89	.135	0.43	.10



Definitions:

Modulus of elasticity { Embankment = E_e ; Foundation = E_f }

Vertical strain = $\delta / d = \epsilon_v$

Horizontal strain = $\Delta L / L = \epsilon_h$

Maximum vertical pressure of embankment = $h\gamma_T = p$

STUDY OF MOVEMENTS
OF ARTICULATED CONDUITS
UNDER EARTH DAMS

MUESER • RUTLEDGE • WENTWORTH & JOHNSTON
CONSULTING ENGINEERS
415 MADISON AVE., NEW YORK 17, N. Y.

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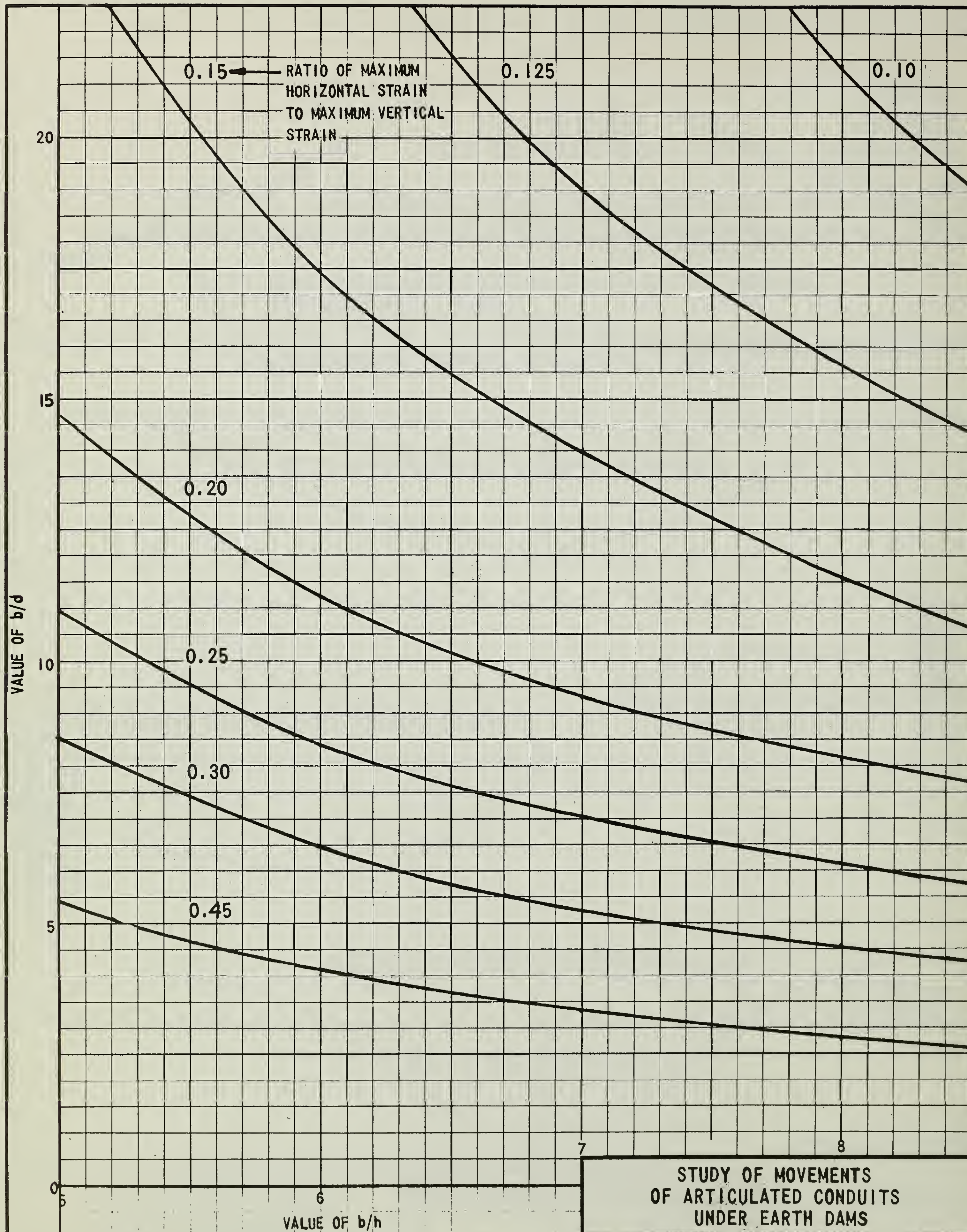

REPRODUCED FROM A PORTION OF PLATE NO. 29
SEPTEMBER 1960 REPORT

STUDY OF MOVEMENTS
OF ARTICULATED CONDUITS
UNDER EARTH DAMS

MUESER·RUTLEDGE·WENTWORTH & JOHNSTON
CONSULTING ENGINEERS
415 MADISON AVE. NEW YORK 17, N. Y.

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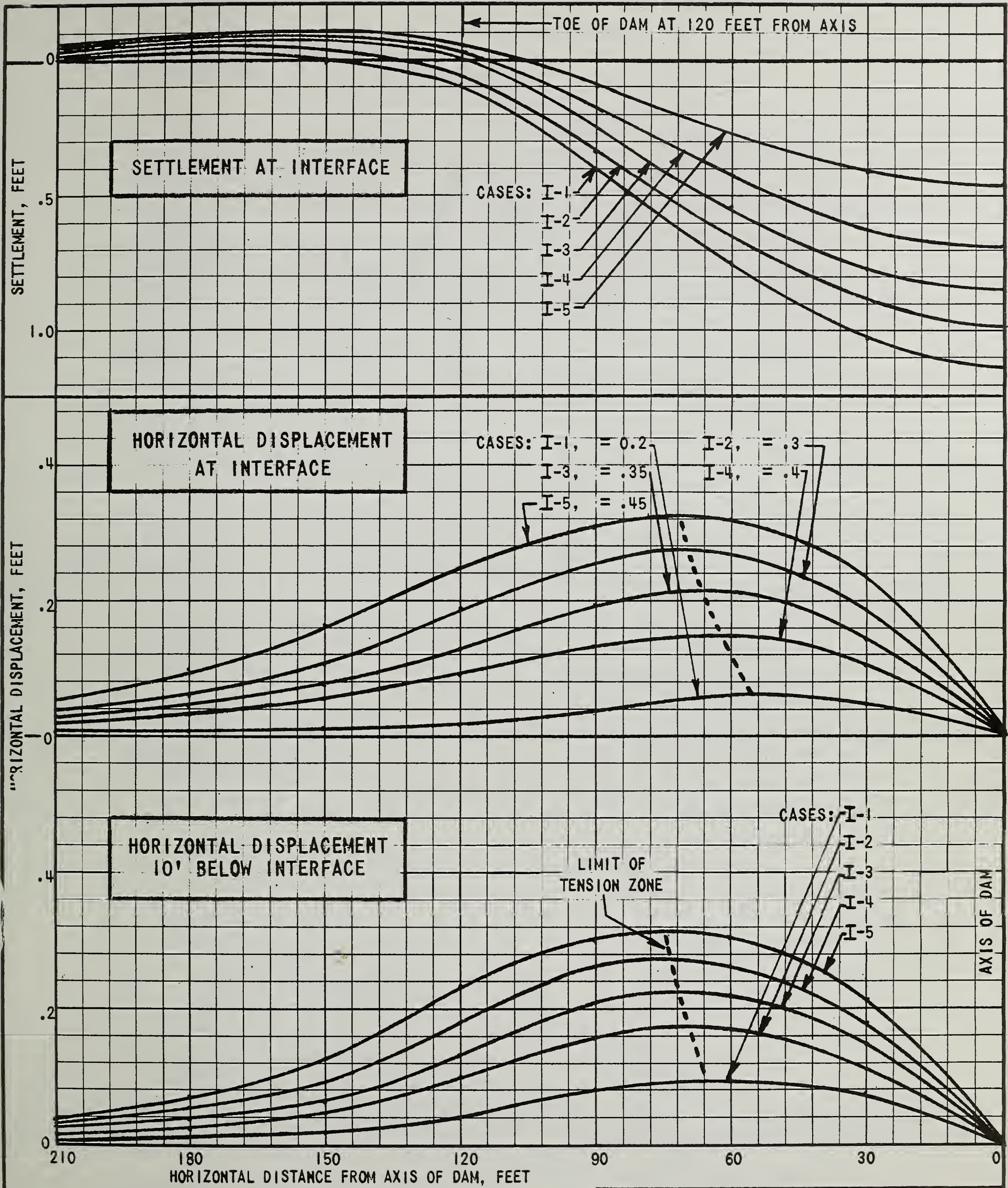
INFLUENCE VALUES
FOR MAXIMUM HORIZONTAL STRAIN

PLATE NO. 2



ASSUMPTIONS:

HEIGHT OF EMBANKMENT = 40' = h
DEPTH OF FOUNDATION = 40' = d
E OF EMBANKMENT = 100 KSF
E OF FOUNDATION = 100 KSF
POISSON'S RATIO = ν = VARIABLE

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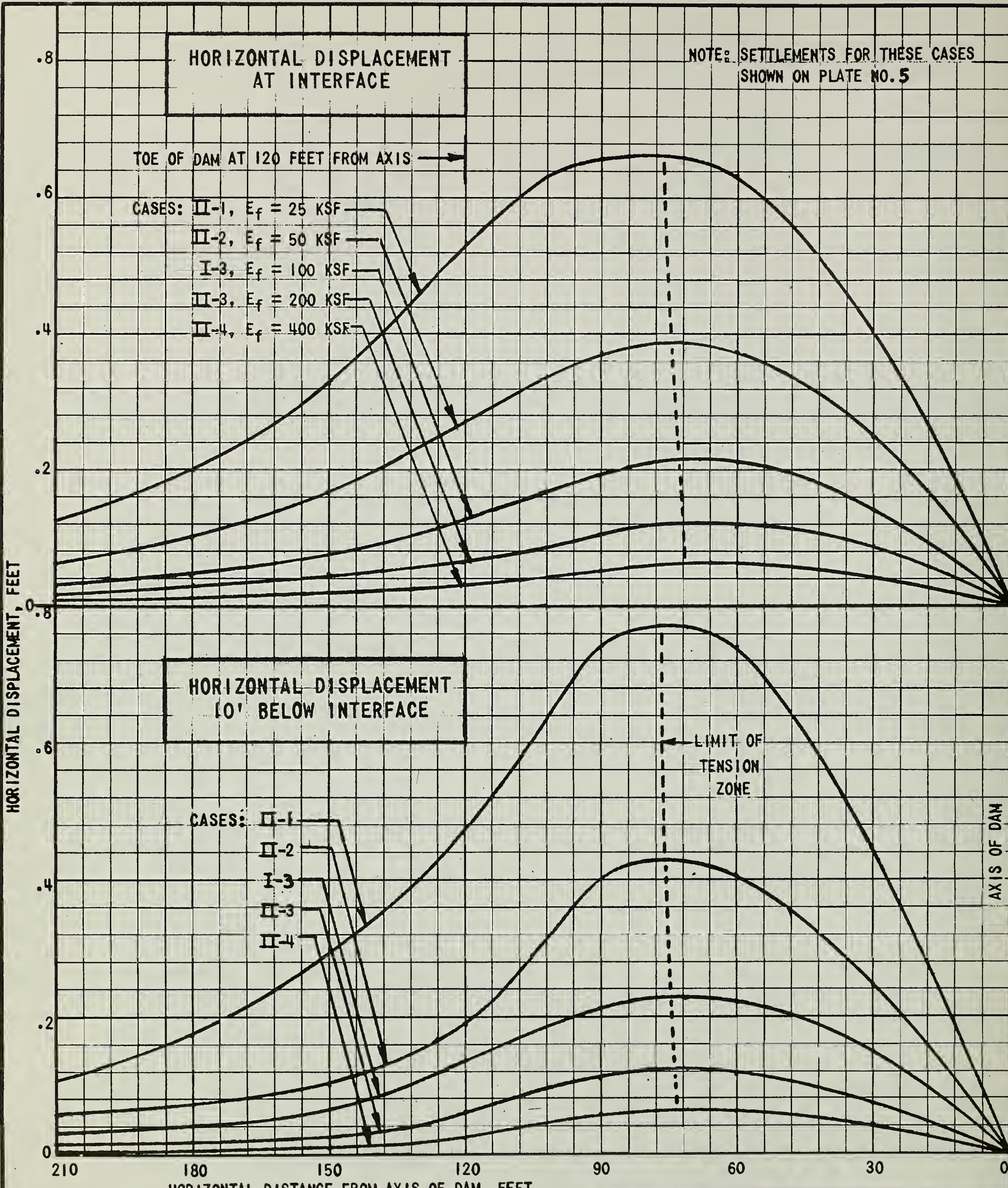
CASES I-1 TO I-5 POISSON'S RATIO VARIABLE			PLATE NO. 3



ASSUMPTIONS:

HEIGHT OF EMBANKMENT = 40' = h
DEPTH OF FOUNDATION = 40' = d
E OF EMBANKMENT = 100 KSF
E OF FOUNDATION = E_f = VARIABLE

POISSON'S RATIO OF EMBANKMENT AND FOUNDATION = μ = 0.35

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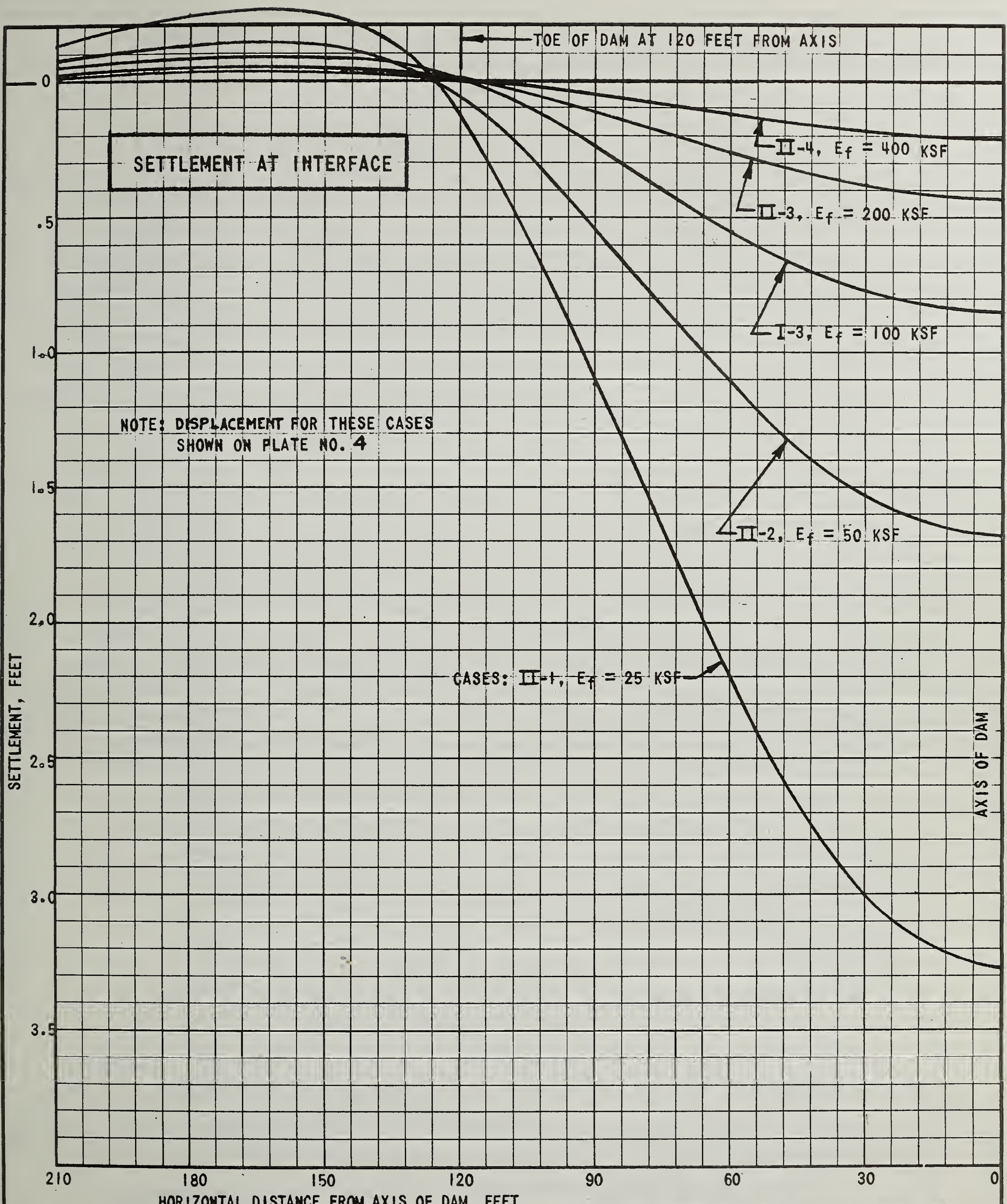
CASES II-1 TO II-4 & I-3
FOUNDATION E VARIABLE

PLATE NO. 4



ASSUMPTIONS:

HEIGHT OF EMBANKMENT = 40' = h
DEPTH OF FOUNDATION = 40' = d
E OF EMBANKMENT = 100 KSF
E OF FOUNDATION = E_f = VARIABLE
POISSON'S RATIO OF EMBANKMENT AND FOUNDATION = ν = 0.35

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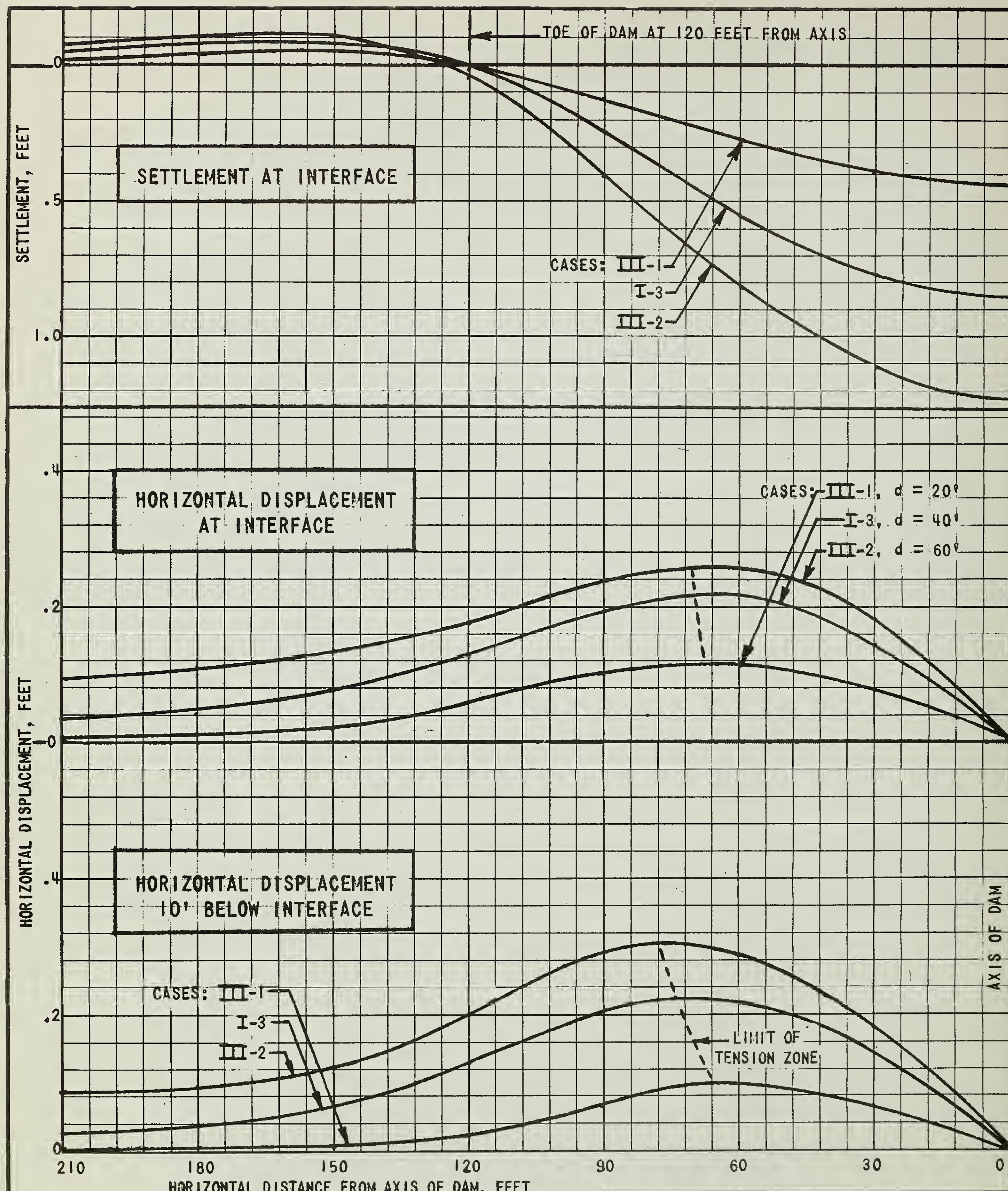
CASES II-1 TO II-4 & I-3
FOUNDATION E VARIABLE

PLATE NO. 5



ASSUMPTIONS:

HEIGHT OF EMBANKMENT = 40' = h
DEPTH OF FOUNDATION = d = VARIABLE
E OF EMBANKMENT = 100 KSF
E OF FOUNDATION = 100 KSF

POISSON'S RATIO OF EMBANKMENT AND FOUNDATION = ν = 0.35

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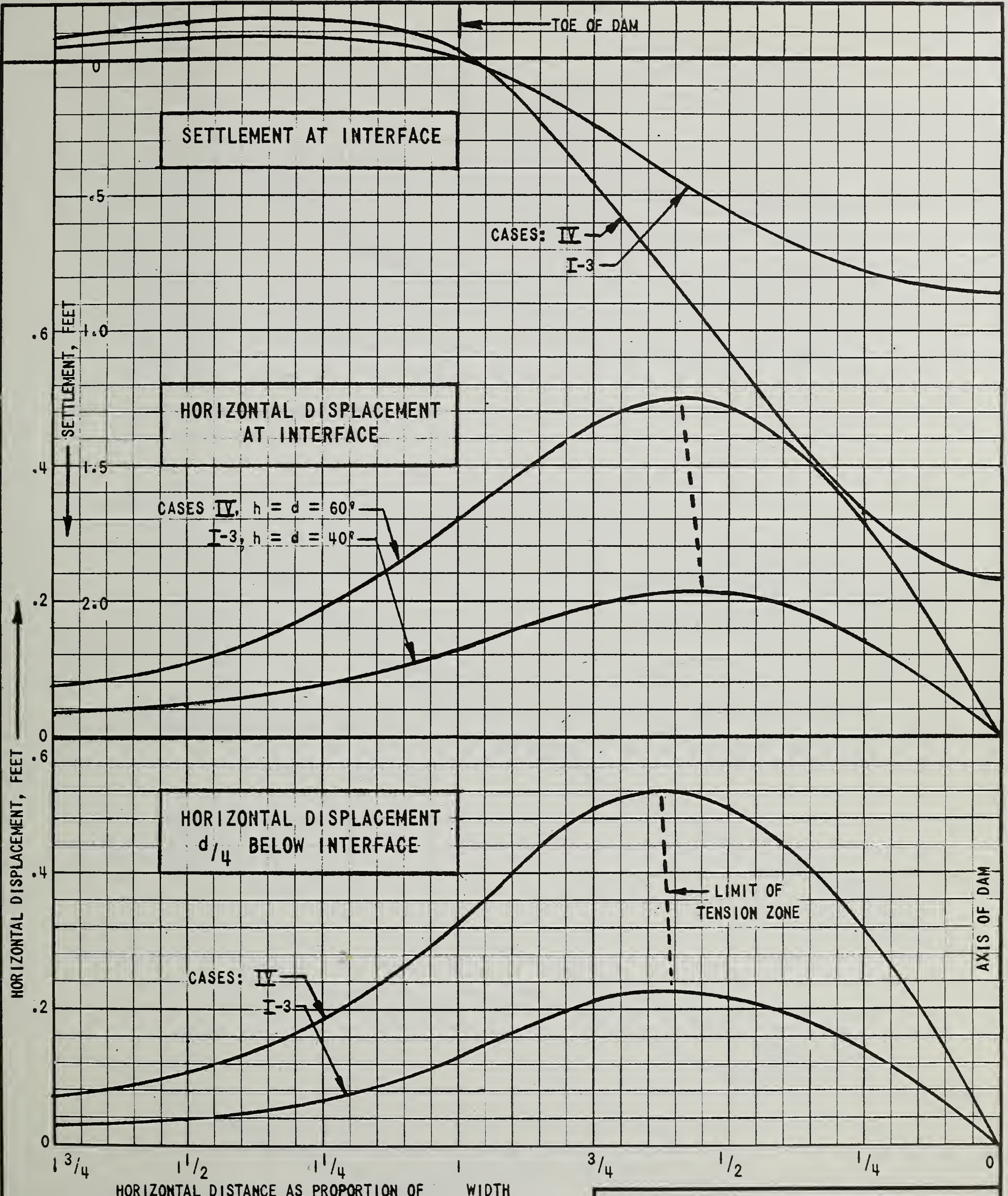
CASES III-1, III-2 & I-3 FOUNDATION DEPTH VARIABLE	PLATE NO. 6



ASSUMPTIONS:

E OF EMBANKMENT = 100 KSF

E OF FOUNDATION = 100 KSF

POISSON'S RATIO OF EMBANKMENT AND FOUNDATION = ν = 0.35

HEIGHT OF EMBANKMENT = h = d = DEPTH OF FOUNDATION

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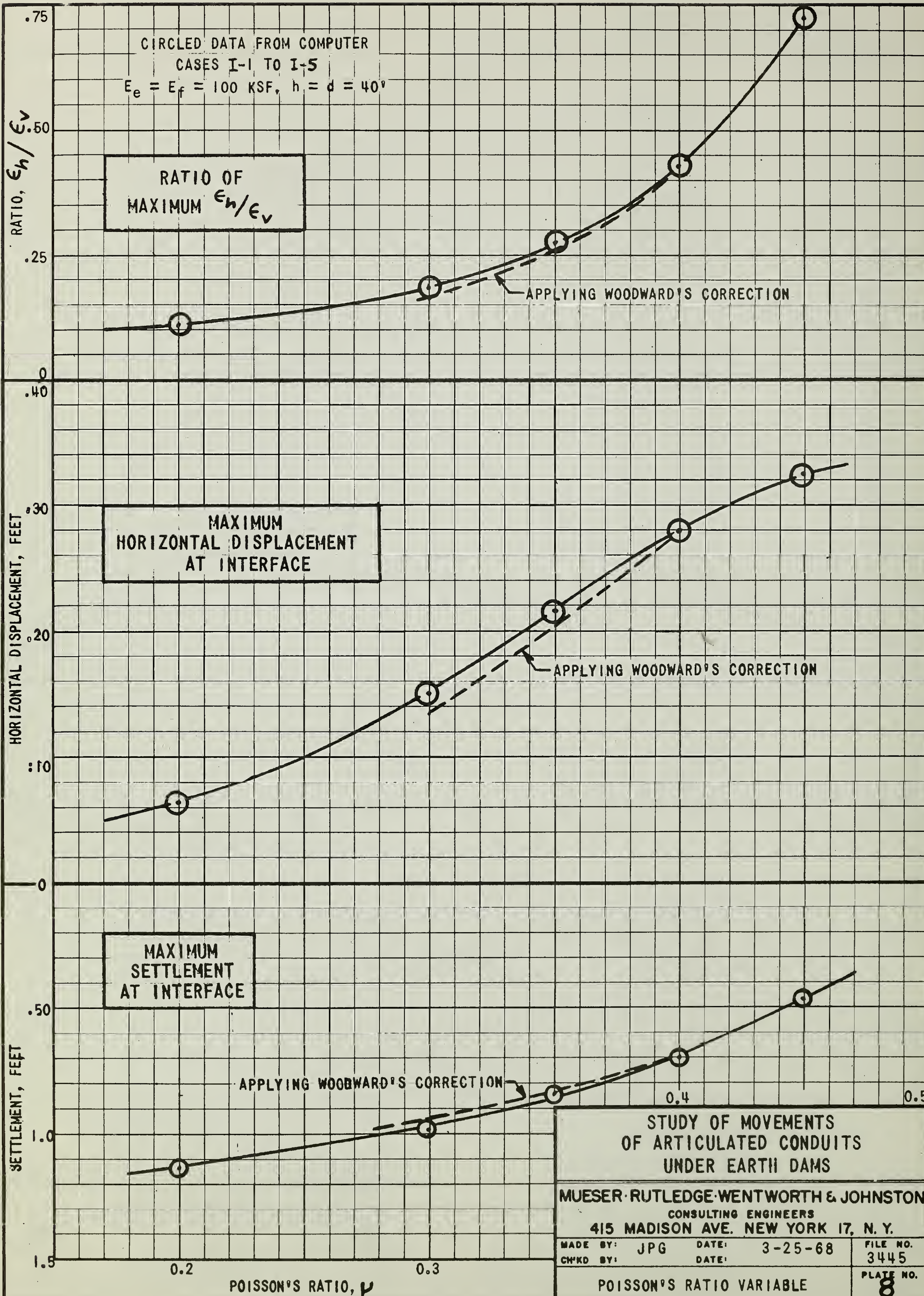
CASES IV AND I-3 SCALE IS VARIABLE	PLATE NO. 7


CIRCLED DATA FROM COMPUTER
CASES I-1 TO I-5
$E_e = E_f = 100$ KSF, h = d = 40'
RATIO, ϵ_h/ϵ_v
.75
.50
.25
0
RATIO OF MAXIMUM ϵ_h/ϵ_v
APPLYING WOODWARD'S CORRECTION
HORIZONTAL DISPLACEMENT, FEET
.40
.30
.20
.10
0
MAXIMUM HORIZONTAL DISPLACEMENT AT INTERFACE
APPLYING WOODWARD'S CORRECTION
SETTLEMENT, FEET
.50
1.0
1.5
MAXIMUM SETTLEMENT AT INTERFACE
APPLYING WOODWARD'S CORRECTION
0.2
0.3
0.4
0.5
POISSON'S RATIO, μ
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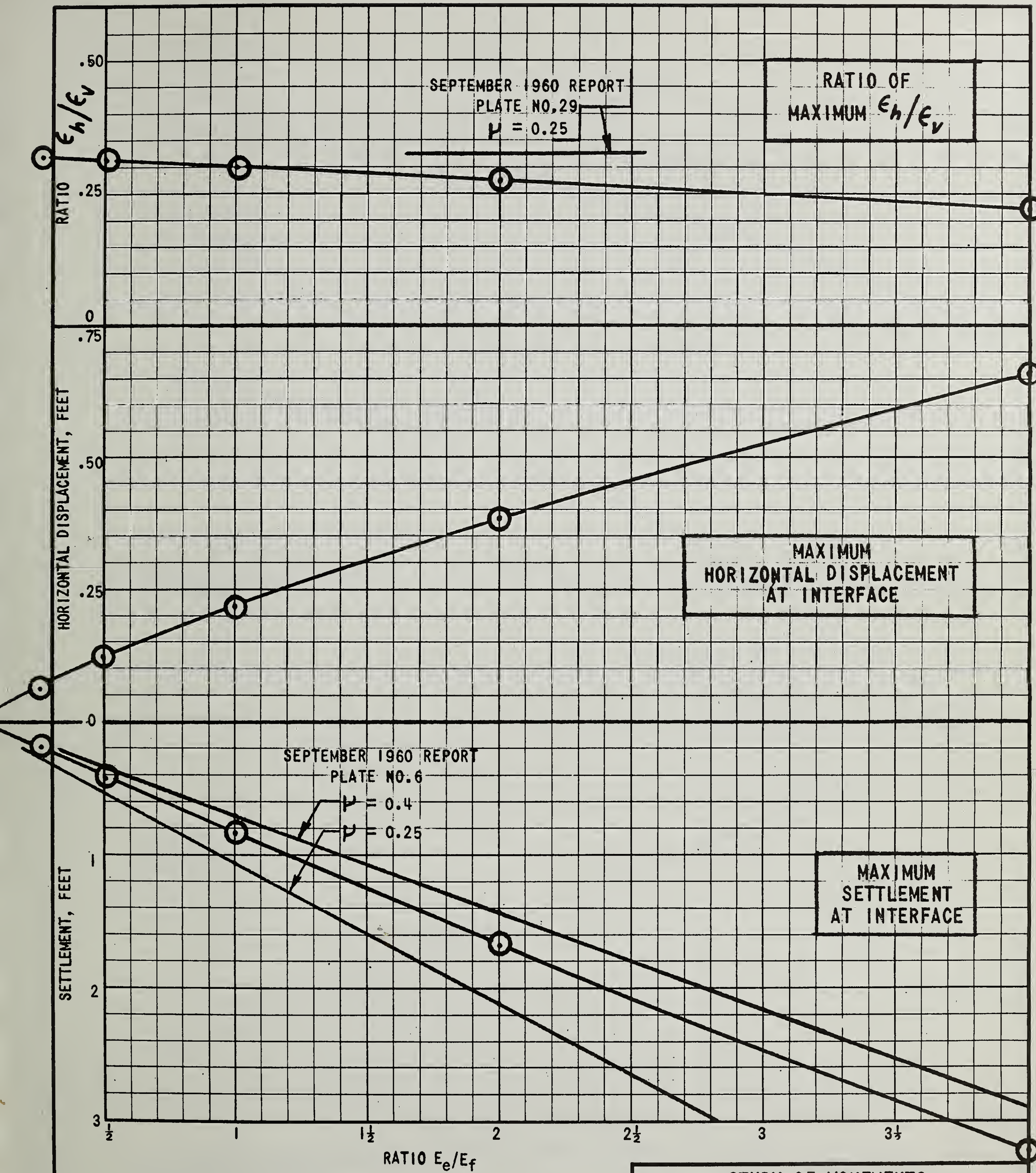
POISSON'S RATIO VARIABLE
PLATE NO. 8



CIRCLED DATA FROM COMPUTER CASES II-1 TO II-4 AND I-3
$\mu = 0.35$, $E_e = 100$ KSF, $h = d = 40'$
E_f = FOUNDATION ELASTIC MODULUS

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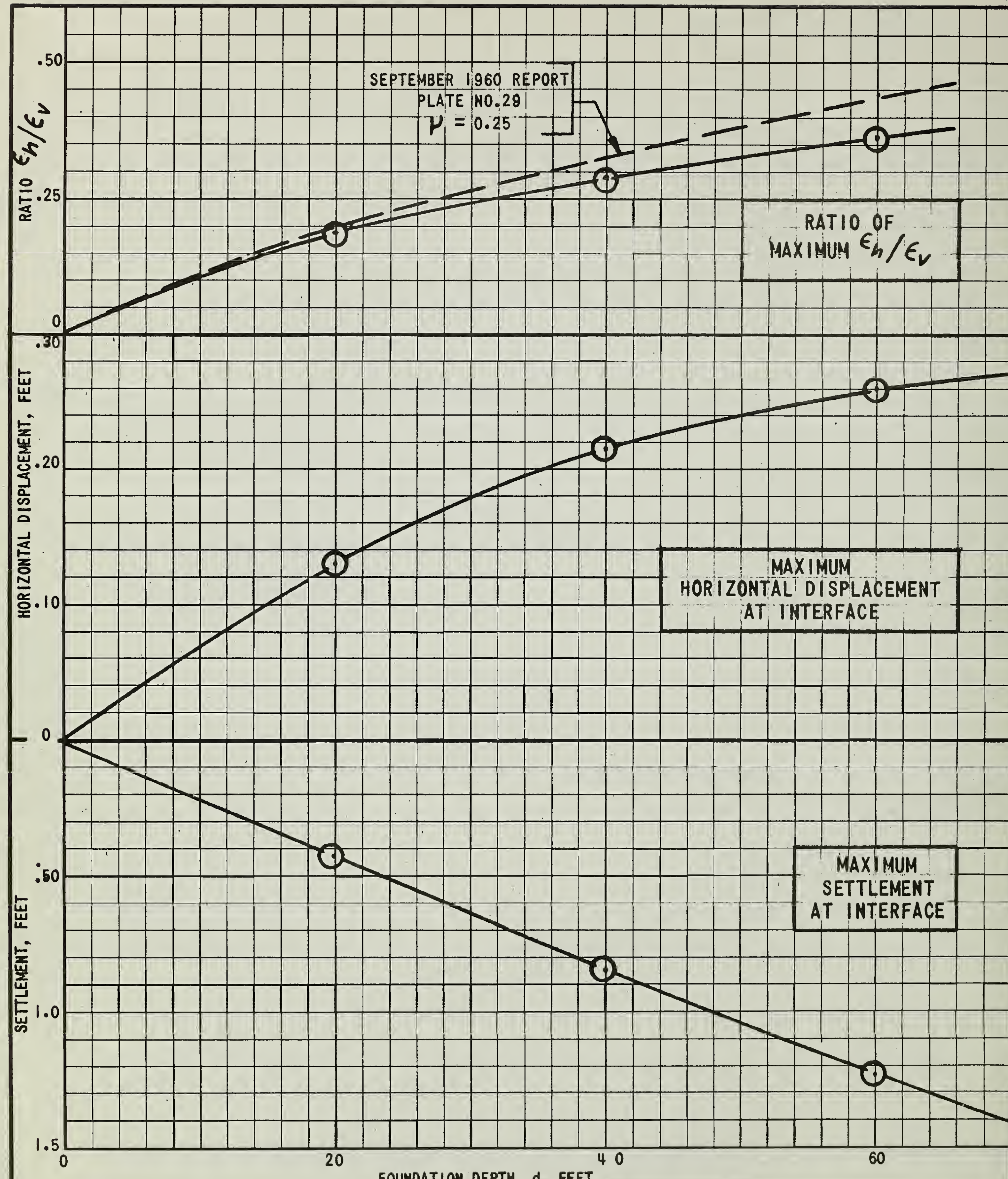
FOUNDATION E VARIABLE

PLATE NO. 9



CIRCLED DATA FROM COMPUTER CASES III-1, III-2 AND I-3
ν = 0.35, $E_e = E_f$ = 100 KSF, h = 40'

STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS

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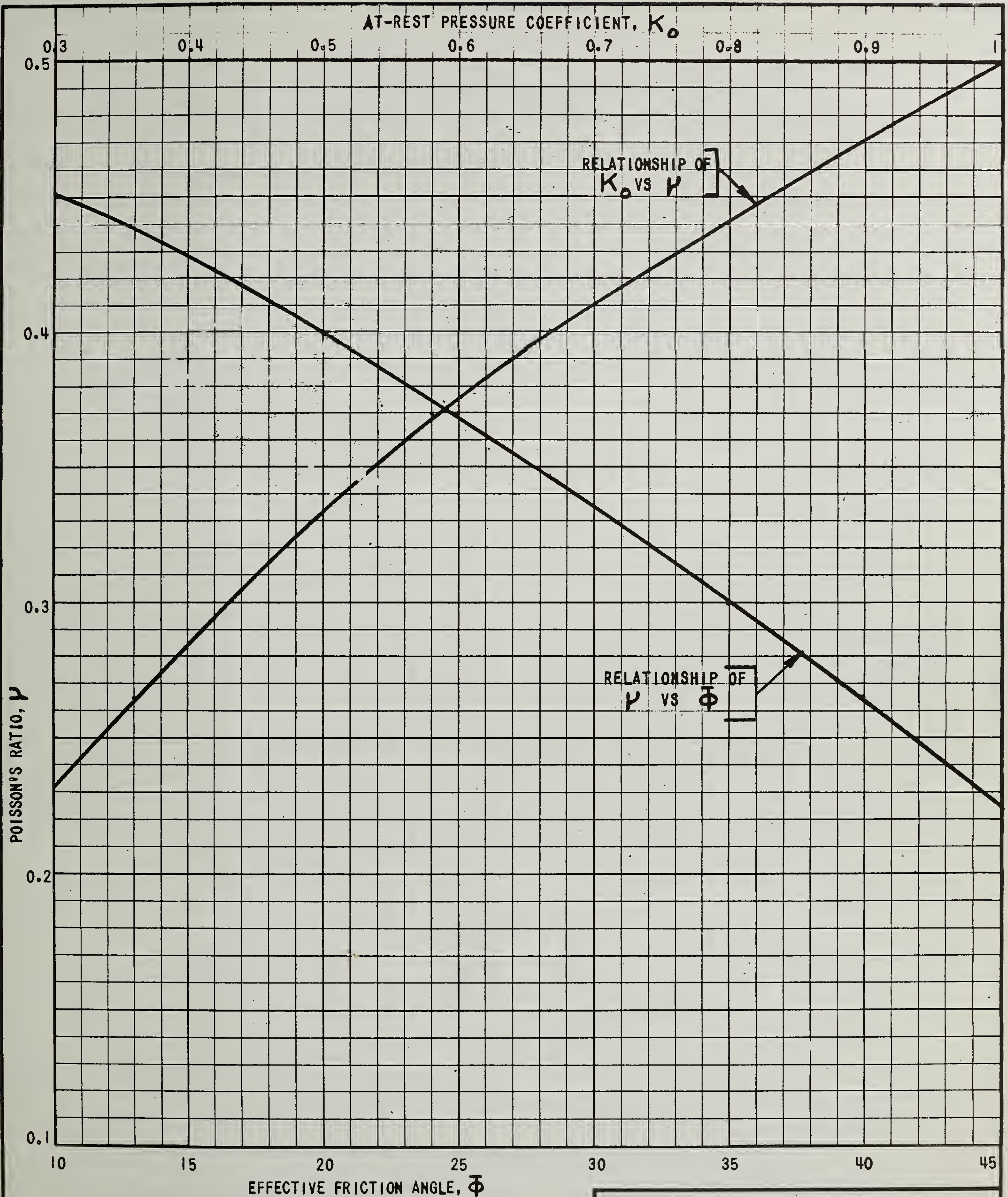
FOUNDATION DEPTH VARIABLE — PLATE NO. 10



RELATIONSHIPS:

$$K_o = 1 - \sin\bar{\Phi} \qquad K_o = \frac{\mu}{1-\mu}$$

$$\sin\bar{\Phi} = \left[\frac{1-2\mu}{1-\mu}\right]$$

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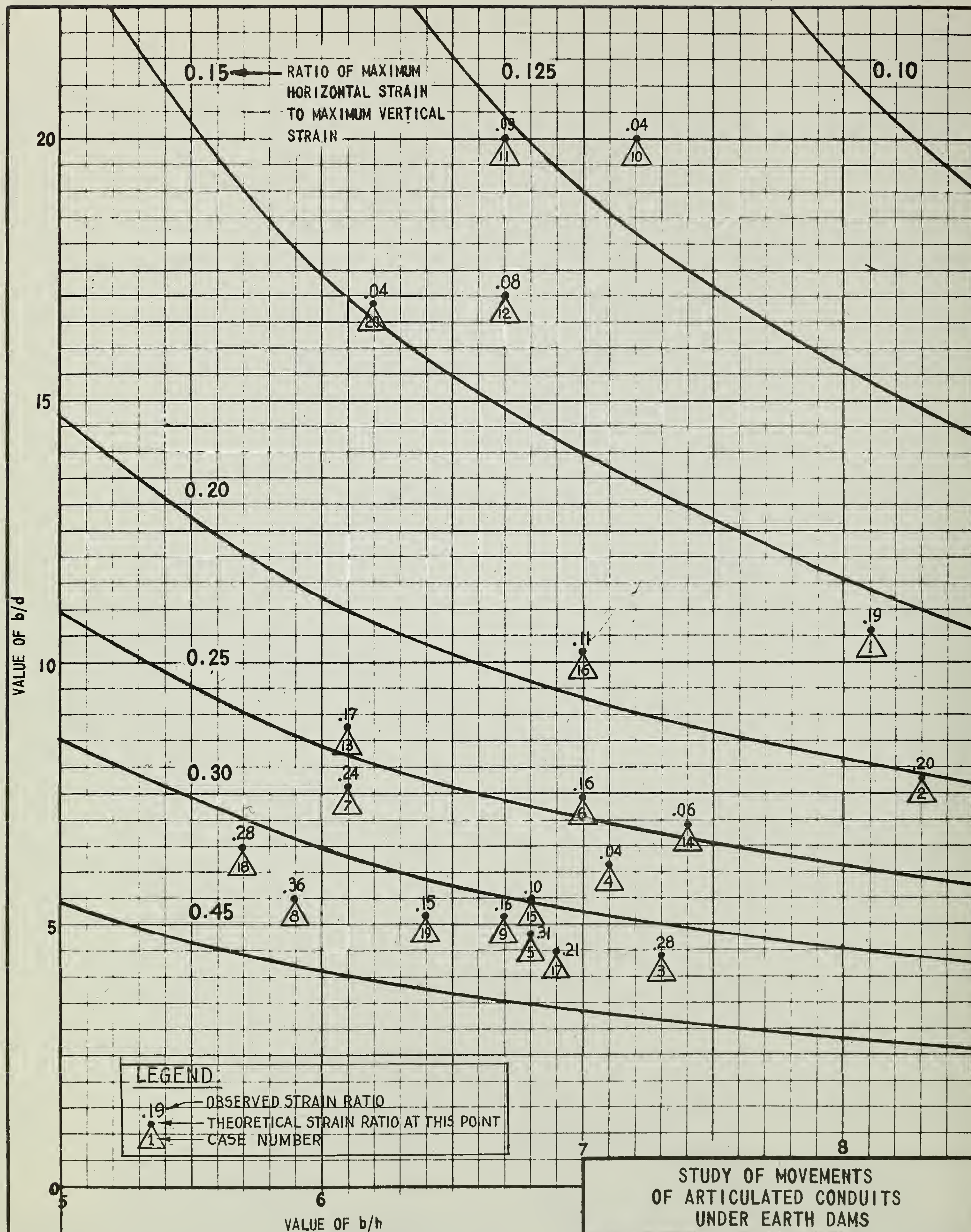
VARIATION OF $\bar{\Phi}$ & K_o WITH POISSON'S RATIO				PLATE NO. 11



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INFLUENCE VALUES FOR MAXIMUM HORIZONTAL STRAIN		PLATE NO. 12	

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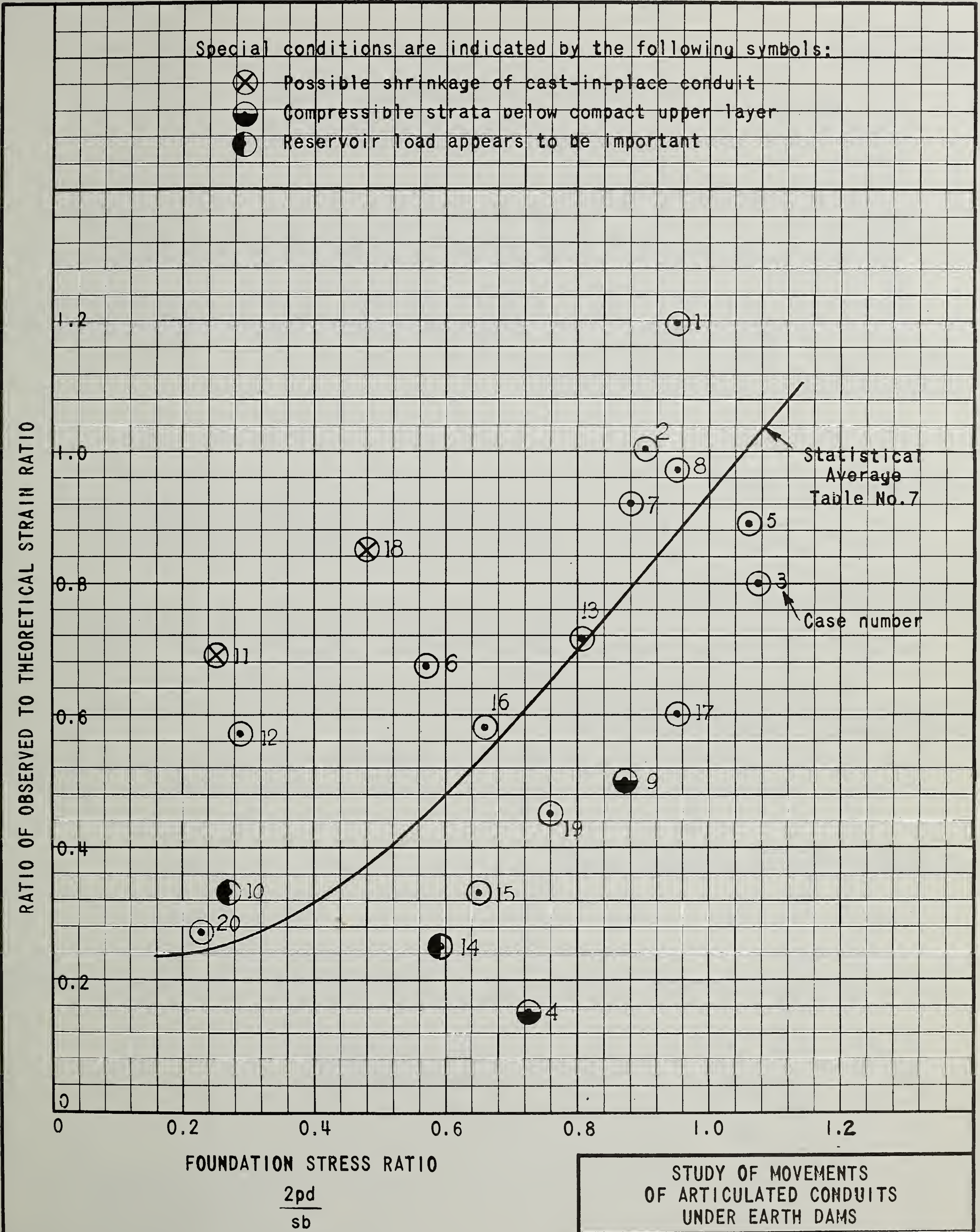

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INFLUENCE OF FOUNDATION STRESS RATIO

PLATE NO. 13

Definitions

$\sigma_x \ \sigma_y \ \sigma_z$ = orthogonal principal stresses

ϵ_z = unit vertical strain observed in test

μ = Poisson's ratio

σ_z/ϵ_z = Apparent modulus of elasticity in test = E_a

Basic E value = E_b, determined from unconfined compression or triaxial test with σ_x = constant

Unconfined Compression

$$E_b = \sigma_z/\epsilon_z$$

Triaxial Shear

$$E_b = (\sigma_z - \sigma_c)/\epsilon_z \quad \begin{cases} \text{when } \sigma_c \text{ is held constant} \\ \sigma_c = \text{chamber pressure} \end{cases}$$

Isotropic Consolidation

$$E_{ai} = \sigma_c/\epsilon_z \qquad E_b = \frac{\sigma_c}{\epsilon_z}(1-2\mu) \qquad E_b = E_{ai}(1-2\mu)$$

Confined Consolidation

$$E_{ac} = \sigma_z/\epsilon_z \qquad E_b = \frac{\sigma_z}{\epsilon_z}\left(1-\frac{2\mu^2}{1-\mu}\right) \qquad E_b = E_{ac}\left(1-\frac{2\mu^2}{1-\mu}\right)$$

Plane Strain Shear Test

$$E_{ap} = \sigma_z/\epsilon_z \qquad E_b = \frac{\sigma_z}{\epsilon_z}(1-\mu^2) \qquad E_b = E_{ap}(1-\mu^2)$$

Ratio of apparent modulus, confined to isotropic consol.

$$\frac{E_{ac}}{E_{ai}} = \frac{(1-\mu)}{(1+\mu)} \qquad E_{ac} = E_{ai}\left(\frac{1-\mu}{1+\mu}\right)$$

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RELATIONSHIPS BETWEEN TEST VALUES OF ELASTIC MODULUS		PLATE NO. 14

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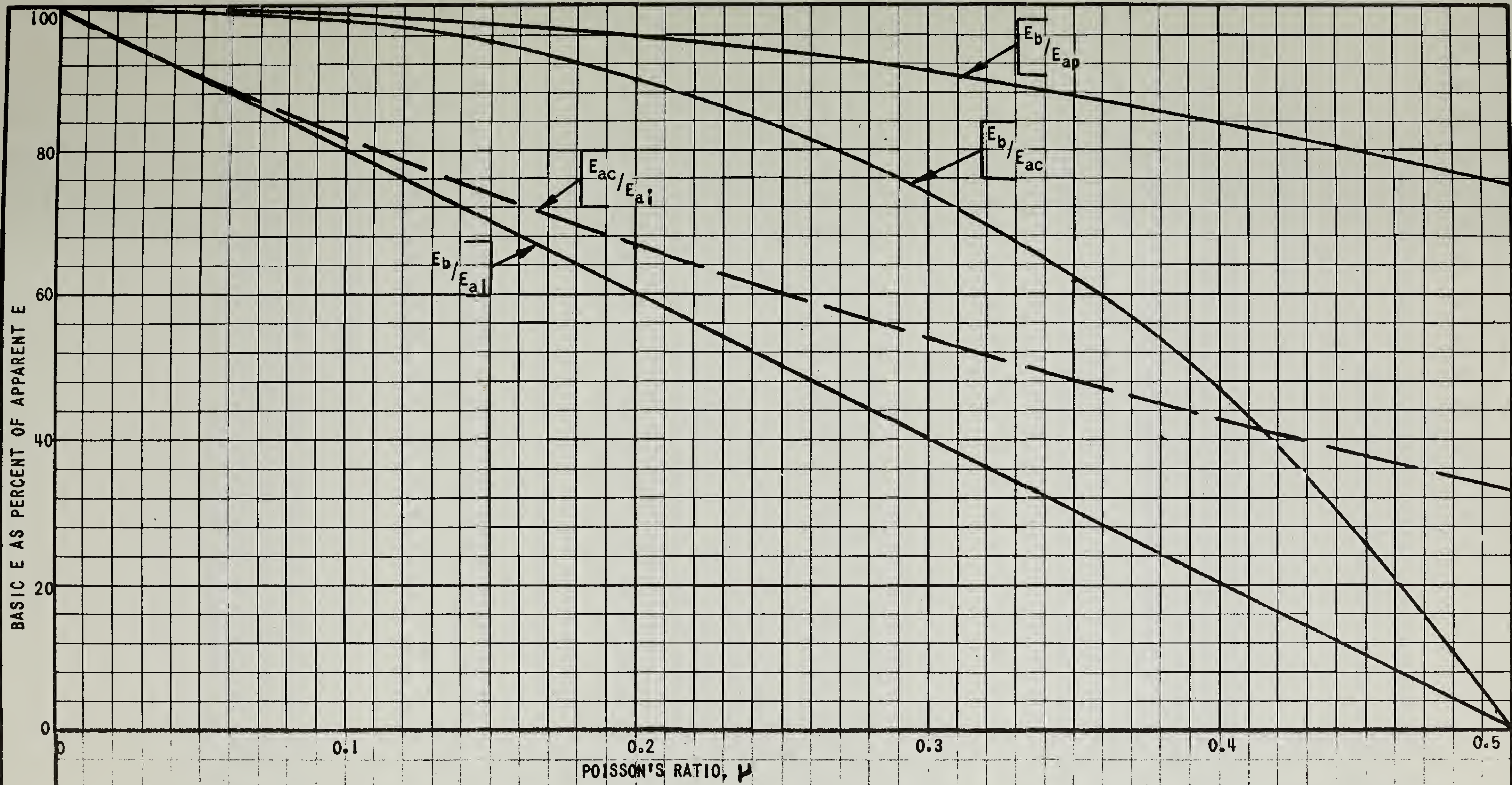

DEFINITIONS:

E_b = BASIC E = STRESS/STRAIN MODULUS DETERMINED FROM UNCONFINED COMPRESSION OR TRIAXIAL TEST WITH σ_3 CONSTANT.

E_a = APPARENT E = STRESS/STRAIN MODULUS DETERMINED FROM ISOTROPIC OR CONFINED CONSOLIDATION OR FROM PLANE STRAIN SHEAR TEST.

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VARIATION OF E WITH POISSON'S RATIO	PLATE NO. 15

APPENDIX A
EVALUATION OF CONDUIT DISPLACEMENTS BY FINITE ELEMENT METHOD

A.1 Introduction

The displacements sustained by the foundation-embankment system under the loads imposed by the embankment material have been ascertained by a computer-evaluated finite element analysis. This appendix describes the finite element method, application of the method to the problem under study, formulation of the computer program, and results obtained with this program. Articles describing the use of the finite element method in plane strain problems involving soil masses regarded as elastic materials are listed in separate sections of the references.

A.2 Basic Assumptions

The foundation and embankment materials were assumed to be isotropic and linearly elastic. These assumptions are warranted in view of the lack of appropriate values for the additional parameters involved in more complex behavior modes. However, they may be inadequate where shear stresses are sufficiently high as to induce significant plastic or other inelastic deformation response in the system. The foundation embankment system was assumed to be subjected to plane strain conditions, that is, strains in the direction perpendicular to the cross-section are considered insignificant. This assumption is a necessary condition for the displacement analysis, but is entirely reasonable for the central portions of those embankments overlying soils without excessive stratigraphic or topographic variation in the direction perpendicular to the section, a condition generally obtaining in the cases being considered. The only restriction on the geometrical configuration of the system is the assumption of a horizontal ground surface and a horizontal lower boundary of the foundation stratum. The base of the foundation layer is considered rough and rigid and no vertical or horizontal movement is permitted at this level. No account was taken of forces induced by the retained water, although these could be readily included in the computer evaluation.

A.3 Cases Considered

Since the finite element method is a relatively recent development, the setting up of a specific program is fraught with "de-bugging" difficulties. In this case a large amount of available computer and personnel time was expended in eliminating program bugs and, as a result the analytical work was limited to a total of twelve cases intended to investigate the influence of the most important parameters. In all cases the embankment was approximated by a symmetrical triangle with side slopes of three horizontal to one vertical. In all cases except No. IV the embankment height at the crest was taken as 40 feet. The modulus of elasticity of the embankment material was assumed to equal 100 kips per square feet. Poisson's ratio was taken equal in the embankment and foundation. For convenience, the embankment total unit weight was taken at 100 pounds per cubic foot. The strains and displacement vary systematically with the scale and weight of the embankment and its stiffness, as explained in Chapter 3, and the above values are chosen merely as representative of the dams under observation.

The various assumptions for the twelve cases are listed in Table No. 2. The first group, Nos. I-1 through I-5, is intended to investigate the effect of changing Poisson's ratio, holding the embankment height and foundation depth equal and the modulus of elasticity constant throughout. The median condition used as a standard in all groups is Case No. I-3 with Poisson's ratio of 0.35. The second group, Nos. II-1 through II-4 and including I-3, is intended to investigate the effect of varying the ratio of foundation to embankment modulus of elasticity, holding embankment height and foundation depth equal and Poisson's ratio constant at 0.35. The third group, Nos. III-1 and III-2 and including I-3, concerns the effect of varying the foundation depth between 20, 30 and 60 feet while holding the embankment height constant at 40 feet. The last case, No. IV, was taken to confirm the scale effect on displacements by assuming embankment height and foundation depth equal to 60 feet.

The displacements obtained from the analysis of the 12 cases are summarized in Plates Nos. 3 through 7. Cases are grouped to show the effect of varying the several parameters. Displacements are plotted for the interface and for a horizontal line in the foundation at a depth of 10 feet below the interface.

A.4 Finite Element Method

The finite element method is an approximation procedure for evaluating the force-displacement behavior of complex structural configurations. In brief, the overall structure is divided into a large number of elements, generally rectangles or triangles, connected solely at their corners, and the displacement behavior of the whole system derived through appropriate combination of relations expressing the stiffness and displacement characteristics of the individual elements. The assumption of uniform stress within elements ensures the necessary continuity along abutting edges. In the following derivation attention is restricted to elements of triangular shape, as these were employed in the present study.

Consideration is thus first given to the behavior of the individual triangle shown in Plate No. A1 (a) . As this is connected to other triangles of a structural situation only at its corner, or nodes, we must ascertain the relation between nodal forces $[S]$ and nodal displacements $[w]$, thus:

$$[S] = [k]\,[w] \tag{1}$$

where $[k]$ represents the stiffness matrix.

It may be shown that the relation between the conventional general strain system $[\epsilon]$ and nodal displacement $[w]$ is as follows:

$$\begin{bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma \end{bmatrix} = \frac{1}{a_j b_k - a_k b_j} \begin{bmatrix} (b_j - b_k) & 0 & b_k & 0 & -b_j & 0 \\ 0 & (a_k - a_j) & 0 & -a_k & 0 & a_j \\ (a_k - a_j) & (b_j - b_k) & -a_k & b_k & a_j & -b_j \end{bmatrix} \begin{bmatrix} u_i \\ v_i \\ u_j \\ v_j \\ u_k \\ v_k \end{bmatrix} \tag{2}$$

or, symbolically:

$$[\epsilon] = [d]\,[w] \tag{2a}$$

The following stress-strain relation is appropriate for the assumed plane strain conditions:

$$\begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix} = \frac{E}{(1+\nu)(1-2\nu)} \begin{bmatrix} (1-\nu) & \nu & 0 \\ \nu & (1-\nu) & 0 \\ 0 & 0 & \left(\frac{1-2\nu}{2}\right) \end{bmatrix} \begin{bmatrix} \epsilon_x \\ \epsilon_y \\ \gamma \end{bmatrix} \tag{3}$$

where: E = modulus of elasticity

ν = Poisson's ratio

or, symbolically:

$$[\sigma] = [e]\,[\epsilon] \tag{3a}$$

The following relation expresses the required nodal forces $[S]$ in terms of the general stress sytem $[\sigma]$:

$$\begin{bmatrix} S_x^i \\ S_y^i \\ S_x^j \\ S_y^j \\ S_x^k \\ S_y^k \end{bmatrix} = \frac{1}{2} \begin{bmatrix} (b_j - b_k) & 0 & (a_k - a_j) \\ 0 & (a_k - a_j) & (b_j - b_k) \\ b_k & 0 & -a_k \\ 0 & -a_k & b_k \\ -b_j & 0 & a_j \\ 0 & a_j & -b_j \end{bmatrix} \begin{bmatrix} \sigma_x \\ \sigma_y \\ \tau_{xy} \end{bmatrix} \qquad (4)$$

or, symbolically: $$[S] = [f]\,[\sigma] \qquad (4a)$$

From expression (2a), (3a) and (4a) we obtain the desired nodal force-nodal displacement, or stiffness relation, thus:

$$[S] = [f]\,[e]\,[d]\,[W] = [k]\,[W] \qquad (5a)$$

where:

$$[k] = [f]\,[e]\,[d]$$

The appropriate matrix manipulations yield the following expression for $[k]$:

$$[k] = \frac{E}{(1+\nu)(1-2\nu)} \cdot \frac{1}{(a_j b_k - a_k b_j)} \begin{bmatrix} (b_j-b_k)^2\alpha + (a_k-a_j)^2\beta & \frac{1}{4}(a_k-a_j)(b_j-b_k) & b_k(b_j-b_k)\alpha - a_k(a_k-a_j)\beta & a_k(b_j-b_k)\frac{\nu}{2} + (a_k-a_j)b_k\beta & -b_j(b_j-b_k)\alpha + a_j(a_k-a_j)\beta & a_j(b_j-b_k)\frac{\nu}{2} - (a_k-a_j)b_j\beta \\ \frac{1}{4}(a_k-a_j)(b_j-b_k) & (a_k-a_j)^2\alpha + (b_j-b_k)^2\beta & b_k(a_k-a_j)\frac{\nu}{2} - a_k(b_j-b_k)\beta & -a_k(a_k-a_j)\alpha + b_k(b_j-b_k)\beta & -b_j(a_k-a_j)\frac{\nu}{2} + a_j(b_j-b_k)\beta & a_j(a_k-a_j)\alpha - b_j(b_j-b_k)\beta \\ b_k(b_j-b_k)\alpha - a_k(a_k-a_j)\beta & b_k(a_k-a_j)\frac{\nu}{2} - a_k(b_j-b_k)\beta & b_k^2\alpha + a_k^2\beta & -\frac{1}{4}a_k b_k & -b_j b_k\alpha - a_j a_k\beta & a_j b_k\frac{\nu}{2} + a_k b_j\beta \\ -a_k(b_j-b_k)\frac{\nu}{2} + (a_k-a_j)b_k\beta & -a_k(a_k-a_j)\alpha + b_k(b_j-b_k)\beta & -\frac{1}{4}a_k b_k & a_k^2\alpha + b_k^2\beta & a_k b_j\frac{\nu}{2} + a_j b_k\beta & -a_j a_k\alpha - b_j b_k\beta \\ -b_j(b_j-b_k)\alpha + a_j(a_k-a_j)\beta & -b_j(a_k-a_j)\frac{\nu}{2} + a_j(b_j-b_k)\beta & -b_j b_k\alpha - a_j a_k\beta & a_k b_j\frac{\nu}{2} + a_j b_k\beta & b_j^2\alpha + a_j^2\beta & -\frac{1}{4}a_j b_j \\ a_j(b_j-b_k)\frac{\nu}{2} - b_j(a_k-a_j)\beta & a_j(a_k-a_j)\alpha - b_j(b_j-b_k)\beta & a_j b_k\frac{\nu}{2} + a_k b_j\beta & -a_j b_k\alpha - b_j b_k\beta & -\frac{1}{4}a_j b_j & a_j^2\alpha + b_j^2\beta \end{bmatrix} \quad (6)$$

where: $\alpha = \left(\frac{1-\nu}{2}\right)$ and $\beta = \left(\frac{1-2\nu}{4}\right)$

The geometrical configuration of the embankment-foundation system makes use of a right-angled triangle element particularly appropriate, as is apparent from Plate No. A2 (a). It can be seen that the selected arrangement involves four triangle orientations. To provide a stable reference system for each of these four basic triangles the right-angle node is restrained against both horizontal and vertical movement and the acute-angle node restrained against vertical movement. The appropriate simplification of the stiffness matrix, expression (6), for the two basic triangle elements forming the upstream segment of the system is then:

$$[k]_u = \frac{E}{(1+\nu)(1-2\nu)} \cdot \frac{1}{h_v h_u} \begin{bmatrix} h_v^2 \alpha & 0 & -h_u h_v \frac{\nu}{2} \\ 0 & h_u^2 \beta & 0 \\ h_u h_v \frac{\nu}{2} & 0 & h_u^2 \alpha \end{bmatrix} \tag{7a}$$

where: h_u = horizontal length of upstream triangle element,

h_v = height of triangle element,

and, for the two basic triangles comprising the downstream segment:

$$[k]_d = \frac{E}{(1+\nu)(1-2\nu)} \cdot \frac{1}{h_d h_v} \begin{bmatrix} h_v^2 \alpha & 0 & h_d h_v \frac{\nu}{2} \\ 0 & h_d^2 \beta & 0 \\ -h_d h_v \frac{\nu}{2} & 0 & h_d^2 \alpha \end{bmatrix} \tag{7b}$$

where: h_d = horizontal length of triangle element.

Consideration must now be given to the displacements sustained by the nodes of the individual basic triangles as a consequence of displacements of nodes within the complete system. This is accomplished from a consideration of Plate No. A1 (b) and (c), in which typical nodes 26 and 17 in general regions of the upstream and downstream systems, respectively, are subject to horizontal and vertical displacements, r_x and r_y. The following relations between the individual triangle node displacements and the system displacements result:

Upstream segment:

$$\begin{bmatrix} u_j \\ u_k \\ v_k \end{bmatrix}_{T} = \begin{bmatrix} -1 & 0 \\ -1 & h_v/h_u \\ 0 & -1 \end{bmatrix}_{T26} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{26} \qquad \begin{bmatrix} u_i \\ u_k \\ v_k \end{bmatrix}_{U} = \begin{bmatrix} 0 & 0 \\ 1 & 0 \\ 0 & 1 \end{bmatrix}_{U26} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{26} \qquad \begin{bmatrix} u_i \\ u_k \\ v_k \end{bmatrix}_{V} = \begin{bmatrix} 1 & 0 \\ 0 & -h_v/h_u \\ 0 & 0 \end{bmatrix}_{V26} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{26}$$

$$\begin{bmatrix} u_j \\ u_k \\ v_k \end{bmatrix}_{K} = \begin{bmatrix} 1 & 0 \\ 0 & -h_v/h_u \\ 0 & 0 \end{bmatrix}_{K26} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{26} \qquad \begin{bmatrix} u_i \\ u_k \\ v_k \end{bmatrix}_{L} = \begin{bmatrix} 0 & 0 \\ 1 & 0 \\ 0 & 1 \end{bmatrix}_{L26} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{26} \qquad \begin{bmatrix} u_i \\ u_k \\ v_k \end{bmatrix}_{M} = \begin{bmatrix} -1 & 0 \\ -1 & h_v/h_u \\ 0 & 0 \end{bmatrix}_{M26} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{26} \tag{8}$$

Downstream segment

$$
\left\{
\begin{aligned}
&\begin{bmatrix} u_j \\ u_k \\ v_k \end{bmatrix}_{Q} = \begin{bmatrix} 1 & 0 \\ 0 & h_v/h_d \\ 0 & 0 \end{bmatrix}_{Q17} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{17}
\qquad
\begin{bmatrix} u_i \\ u_j \\ v_k \end{bmatrix}_{R} = \begin{bmatrix} 0 & 0 \\ 1 & 0 \\ 0 & 1 \end{bmatrix}_{R17} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{17}
\qquad
\begin{bmatrix} u_j \\ u_k \\ v_k \end{bmatrix}_{S} = \begin{bmatrix} -1 & 0 \\ -1 & h_v/h_d \\ 0 & 0 \end{bmatrix}_{S17} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{17}
\\
&\begin{bmatrix} u_i \\ u_j \\ v_k \end{bmatrix}_{H} = \begin{bmatrix} -1 & 0 \\ -1 & h_v/h_d \\ 0 & 0 \end{bmatrix}_{H17} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{17}
\qquad
\begin{bmatrix} u_i \\ u_k \\ v_k \end{bmatrix}_{I} = \begin{bmatrix} 0 & 0 \\ 1 & 0 \\ 0 & 1 \end{bmatrix}_{I17} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{17}
\qquad
\begin{bmatrix} u_i \\ u_j \\ v_j \end{bmatrix}_{J} = \begin{bmatrix} 1 & 0 \\ 0 & h_v/h_d \\ 0 & 0 \end{bmatrix}_{J17} \begin{bmatrix} r_x \\ r_y \end{bmatrix}_{17}
\end{aligned}
\right.
\tag{9}
$$

or, symbolically:

$$[w] = [a][r] \qquad \left\{\begin{matrix} (8a) \\ (9a) \end{matrix}\right.$$

The comprehensive matrix relation between nodal forces and nodal displacements for the entire system essentially comprises an appropriate assemblage of the relations (7a), (7b), (8a) and (9a) for all the triangles comprising the system. This formulation is illustrated for the typical region of the upstream segment shown in Plate No. A1 (b).

The complete stiffness matrix $[K]$ for the constituent elements of the region incorporates the individual stiffness matrices (k), relations (7a), as successive terms on the leading diagonal, as follows:

$$
[K] = \begin{bmatrix}
\bullet & 0 & 0 & 0 & & & & & & & & & & & 0 \\
0 & k_A & 0 & & & & & & & & & & & & \\
0 & 0 & k_B & & & & & & & & & & & & \\
0 & & & k_C & & & & & & & & & & & \\
 & & & & k_D & & & & & & & & & & \\
 & & & & & k_E & & & & & & & & & \\
 & & & & & & \ddots & & & & & & & & \\
 & & & & & & & k_S & & & & & & & \\
 & & & & & & & & k_T & & & & & & \\
 & & & & & & & & & k_U & & & & & \\
 & & & & & & & & & & k_V & & & & 0 \\
 & & & & & & & & & & & k_W & 0 & & 0 \\
 & & & & & & & & & & & 0 & k_X & & 0 \\
0 & & & & & & & & & & 0 & 0 & 0 & & \bullet
\end{bmatrix}
\tag{10}
$$

The comprehensive displacement matrix $[A]$ for the entire system comprises an assemblage of the displacement relations (8a) for each triangle taken in the same order as in the comprehensive stiffness matrix $[K]$, matrix (10). Thus, for the region in Plate No. A1 (b) we obtain the following matrix:

$$
\begin{bmatrix} W_A \\ W_B \\ W_C \\ W_D \\ W_E \\ W_F \\ W_K \\ W_L \\ W_M \\ W_N \\ W_O \\ W_P \\ W_S \\ W_T \\ W_U \\ W_V \\ W_W \\ W_X \end{bmatrix}
=
\begin{bmatrix}
a_{A3} & & & & a_{A11} & a_{A12} & & & & & & & & & & \\
a_{B3} & a_{B4} & & & & a_{B12} & & & & & & & & & & \\
& a_{C4} & & & & a_{C12} & a_{C13} & & & & & & & & & \\
& a_{D4} & a_{D5} & & & & a_{D13} & & & & & & & 0 & & \\
& & a_{E5} & & & & a_{E13} & a_{E14} & & & & & & & & \\
& & a_{F5} & a_{F6} & & & & a_{F14} & & & & & & & & \\
& & & & a_{K11} & & & & a_{K25} & a_{K26} & & & & & & \\
& & & & a_{L11} & a_{L12} & & & & a_{L26} & & & & & & \\
& & & & & a_{M12} & & & & a_{M26} & a_{M27} & & & & & \\
& & & & & a_{N12} & a_{N13} & & & & a_{N27} & & & & & \\
& & & & & & a_{O13} & & & & a_{O27} & a_{O28} & & & & \\
& & & & & & a_{P13} & a_{P14} & & & & a_{P28} & & & & \\
& & & & & & & & a_{S25} & & & & a_{S32} & a_{S33} & & \\
& 0 & & & & & & & a_{T25} & a_{T26} & & & & a_{T33} & & \\
& & & & & & & & & a_{U26} & & & & a_{U33} & a_{U34} & \\
& & & & & & & & & a_{V26} & a_{V27} & & & & a_{V34} & \\
& & & & & & & & & & a_{W27} & & & & a_{W34} & a_{W35} \\
& & & & & & & & & & a_{X27} & a_{X28} & & & & a_{X35}
\end{bmatrix}
\begin{bmatrix} r_3 \\ r_4 \\ r_5 \\ r_6 \\ r_{11} \\ r_{12} \\ r_{13} \\ r_{14} \\ r_{25} \\ r_{26} \\ r_{27} \\ r_{28} \\ r_{32} \\ r_{33} \\ r_{34} \\ r_{35} \end{bmatrix}
\qquad (11)
$$

or, symbolically:

$$[W] = [A][R] \qquad (11a)$$

The stiffness matrix $[K]$ representing the stiffness of the entire system acting as a system is then obtained from matrices (10) and (11) by the typical coordinate transformation procedure of matrix algebra, thus:

$$[K] = [A]^T [K] [A] \qquad (12)$$

As in the matrix expression (1), in which the stiffness matrix $[k]$ for an individual triangle relates the nodal forces and nodal displacements for that triangle, so does the stiffness $[K]$ relate the nodal forces $[S]$ and nodal displacements for the complete element system acting as a system:

$$[S] = [K] [R] \qquad (13)$$

The nodal forces $[S]$ in relation (13), are the applied and/or body forces acting on the nodes, which must be listed in the same order as the nodes in the right hand side of expression (11). The force at any node is taken equal to the product of the embankment unit weight and that area having a perimeter connecting points midway between that node and the immediately adjacent nodes. In the present study the nodal forces $[S]$ are specified and the nodal displacement $[R]$ the unknowns required to be evaluated. Thus, the relation (13) must be inverted to yield the following relation:

$$[R] = [K]^{-1} [S] \qquad (14)$$

were $[K]$ is referred to as the flexibility matrix.

The order of the flexibility matrix is invariably so high as to render solution by electronic computation an absolute necessity. The following section outlines the procedure followed in the formulation of the program developed for solution of the subject displacement problem.

A.5 Computer Program, General

The subject displacement problem was programmed for finite element solution using the Fortran IV language and solutions obtained on a Control Data Corporation 3600 computer. Every effort was made to utilize only those features of the Fortran IV language common to both the CDC 3600 computer and the larger IBM machines in order to facilitate usage of the program with those of the latter machines employed by the Soil Conservation Service. It should be noted that the inversion of the stiffness matrix $[K]$ to obtain the flexibility matrix $[K]^{-1}$ was performed with the Control Data Corporation linear inversion program ALLEGRO, the details of which are not publicly available. However, any high precision inversion program may be employed for this final matrix manipulation.

A.6 Computer Program, Basic Approach

The basic approach employed in the program formulation is now outlined. This outline, and the numerous items of clarifying information presented on the program comment cards, should permit an understanding of the steps in the program listing included in Section A.8.

In brief, the computer program comprises a direct evaluation of the terms in the stiffness matrix $[K]$ and load matrix $[S]$; subsequent inversion of the former matrix, employing ALLEGRO, to yield the flexibility matrix $[K]^{-1}$ and multiplication with the latter matrix, in accordance with expression (14), giving directly the desired nodal displacements $[R]$. The procedure employed for the direct evaluation of the terms in the stiffness matrix $[K]$ is conveniently presented with reference to the typical region of the upstream segment of the subgrade-embankment system shown in Plate No. A1(b). It will be recalled that matrices (10) and (11) respectively represent the stiffness and displacement matrices for the elements comprising this typical region. The substitution of these last-mentioned matrices into expression (12) and their appropriate manipulation yields the stiffness matrix $[K]$. A typical row in the stiffness matrix for a node centrally located in the region under consideration, say node number 26, is as follows. The externally located nodes clearly cannot be fully specified:

$$
\begin{array}{cccccccc}
\text{Nodes:} & (11) & (12) & (25) & (26) & (27) & (33) & (34)
\end{array}
$$

$$
[K] = \left[\begin{array}{ccccccc}
 & & & [a]^T_{K26}[k]_K[a]_{K26} & & & \\
 & & & + & & & \\
 & & & [a]^T_{L26}[k]_L[a]_{L26} & & & \\
 & & & + & & & \\
[a]^T_{K26}[k]_K[a]_{K11} & [a]^T_{L26}[k]_L[a]_{L12} & [a]^T_{K26}[k]_K[a]_{K25} & [a]^T_{M26}[k]_M[a]_{M26} & [a]^T_{M26}[k]_M[a]_{M27} & [a]^T_{T26}[k]_T[a]_{T33} & [a]^T_{U26}[k]_U[a]_{U34} \\
+ & + & + & + & + & + & + \\
[a]^T_{L26}[k]_L[a]_{L11} & [a]^T_{M26}[k]_M[a]_{M12} & [a]^T_{T26}[k]_T[a]_{T25} & [a]^T_{T26}[k]_T[a]_{T26} & [a]^T_{V26}[k]_V[a]_{V27} & [a]^T_{U26}[k]_U[a]_{U33} & [a]^T_{V26}[k]_V[a]_{V34} \\
 & & & + & & & \\
 & & & [a]^T_{U26}[k]_U[a]_{U26} & & & \\
 & & & + & & & \\
 & & & [a]^T_{V26}[k]_V[a]_{V26} & & &
\end{array}\right] \tag{15}
$$

Some simplification of the above typical matrix is possible on the basis of previous considerations. Thus, all the stiffness matrix terms $[k]$ are necessarily equal to the matrix $[K]_u$ specified in relation (7a) as follows:

$$
[k]_{K} = [k]_{L} = [k]_{M} = [k]_{T} = [k]_{U} = [k]_{V} = [k]_u \tag{16}
$$

Further, reference to relation (8) establishes the following:

$$
\begin{aligned}
&[a]_{K26} = [a]_{L11} = [a]_{T25} = [a]_{V26} = [a]_{M27} = [a]_{U34} = \begin{bmatrix} 1 & 0 \\ 0 & -h_v/h_u \\ 0 & 0 \end{bmatrix} = [a]_{UACU} \\
&[a]_{K11} = [a]_{L26} = [a]_{S25} = [a]_{M12} = [a]_{N27} = [a]_{U26} = [a]_{T33} = \begin{bmatrix} 0 & 0 \\ 1 & 0 \\ 0 & 1 \end{bmatrix} = [a]_{UINT} \\
&[a]_{L12} = [a]_{M26} = [a]_{K25} = [a]_{T26} = [a]_{V27} = [a]_{U33} = \begin{bmatrix} -1 & 0 \\ -1 & -h_v/h_u \\ 0 & -1 \end{bmatrix} = [a]_{URIT}
\end{aligned} \tag{17}
$$

The subscripts UACU, UINT, and URIT refer to the nature of the angle of the specified Upstream triangle, either ACUTE, INTermediate, or RIghT-angle, contacting the specified node.

Thus, each row in the stiffness matrix $[K]$ may be formed in the manner of relation (15) for the typical node 26, and the terms thereby involved then replaced by the matrices (16) and (17) with the appropriate substitutions for the variables involved. Such substitution reveals that there are nine distinct terms of the form $[a]^T[k][a]$ for both the upstream and downstream sides for each layer of the system, where a layer is the area between adjoining horizontal lines.

The nine terms for the upstream side are as follows:

$$\begin{array}{ccc}
[a]^T_{UACU}[k]_U[a]_{UACU} & [a]^T_{UINT}[k]_U[a]_{UACU} & [a]^T_{URIT}[k]_U[a]_{UACU} \\
[a]^T_{UACU}[k]_U[a]_{UINT} & [a]^T_{UINT}[k]_U[a]_{UINT} & [a]^T_{URIT}[k]_U[a]_{UINT} \\
[a]^T_{UACU}[k]_U[a]_{URIT} & [a]^T_{UINT}[k]_U[a]_{URIT} & [a]^T_{URIT}[k]_U[a]_{URIT}
\end{array} \qquad (18)$$

The terms in the stiffness matrix $[K]$ may accordingly be obtained by summing the appropriate terms of (18), or the corresponding downstream terms. The evaluation of the body forces to be applied to the nodes throughout the embankment then provides the right hand side for multiplication with $[K]$.

A.7 Computer Program, Specification of Foundation-Embankment System

The system shown in Plate No. A2 (a) is the most general system that can be analyzed with the program. The configuration actually employed for this analysis is shown on Plate No. A2 (b). Besides the limitation imposed by the memory space of the selected computer, the only restrictions on the geometrical configuration are as follows:

(1) The constituent elements of the system must be right-angled triangles, with the hypotenuse sloping down to the left within the upstream (or left) side and down to the right within the downstream (or right) side.

(2) The horizontal lengths of the upstream and downstream triangles must be respectively uniform.

(3) The vertical heights of all the triangles in any layer must be uniform.

(4) The embankment may have only one berm on either side.

(5) The width of the embankment top may either be zero or equal to the horizontal length of the downstream triangles.

(6) The lower boundary of the foundation is assumed to be rigidly restrained.

(7) The physical properties of the soil within a specific layer must be uniform.

The following variables must accordingly be specified for each layer of the system (to repeat, a layer denotes the area between adjoining horizontal lines of the element arrangement):

(1) Number of triangles of selected uniform horizontal lengths, thereby specifying overall horizontal dimensions of system.

(2) Vertical height of triangles, thereby specifying overall vertical dimensions of system.

(3) Density, modulus of elasticity and Poisson's ratio values.

A.8 Computer Program - Program Listing

The following comments are intended as an aid to a detailed study of the complete listing presented on the print-out sheets submitted separately to the Soil Conservation Service:

(1) Dimension statements. The program was prepared to analyze systems involving up to nine layers; consideration of greater numbers will require appropriate revision of those arrays containing the integer 9.

(2) Data statements. The stiffness matrices $[k]_u$ and $[k]_d$ are entered as data statements except that terms comprising ratios of triangles lengths are here specified as zero, to be modified at a later stage.

(3) Write statements. These cover all the variables in the problem.

(4) Execution

(a) The constituent terms of the stiffness matrix $[K]$ of the form $[a]^T[k][a]$, of which the upstream expressions are as given in (18), are evaluated for each layer employing conventional matrix algebra procedure. All the values for each layer are printed.

(b) The node forces are evaluated for each layer and printed in conjunction with the constituent stiffness terms in (a).

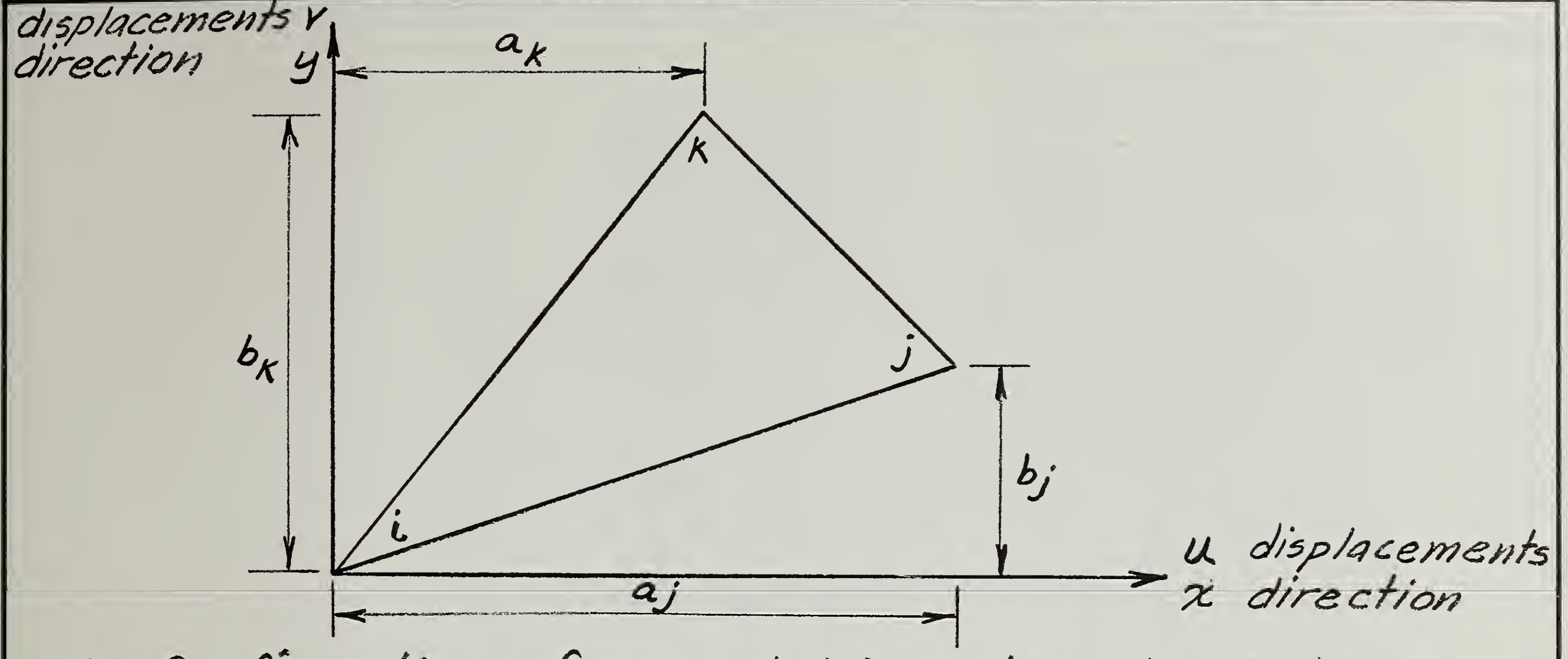
(c) The complete terms in the stiffness matrix $[K]$ are then evaluated by proceeding upstream to downstream along each layer in turn, starting with the lowest. A reference system is employed in which the numbers corresponding to the vertical and horizontal directions at each node are respectively equal to twice the number of the node, numbering off in the above manner, and one less than that number.

(d) Node forces are incorporated at the appropriate nodes.

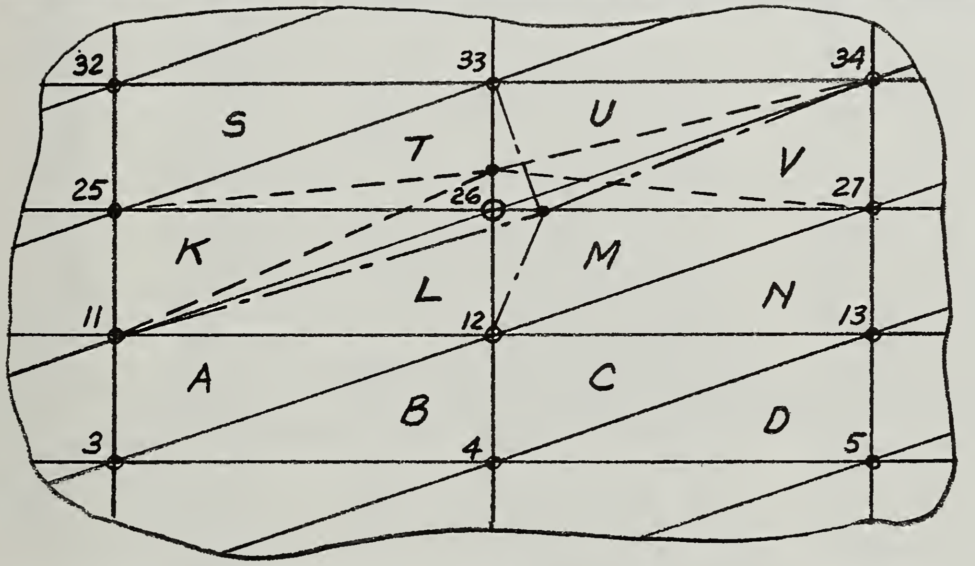
(e) The terms of the stiffness matrix $[K]$ are finally listed column by column in the manner necessary for input to the Control Data Corporation inversion program ALLEGRO. As no term in either the matrix or right hand side (node forces) may exceed unity for manipulation with this program the values in each column have been divided by the largest value therein. The output from ALLEGRO must, of course, be appropriately modified by these largest values.

A.9 Computer Program, Utilization

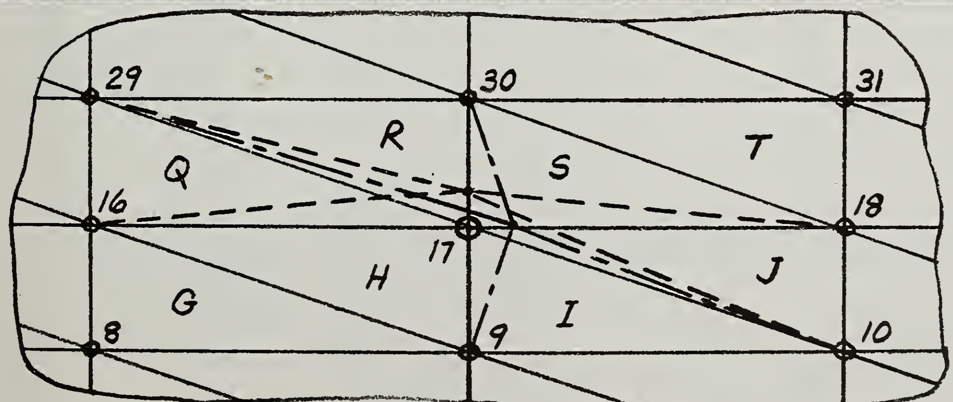
The program provides correct solutions consistent with the assumptions inherent in the fundamental derivation of the finite element system. However, despite the uniformity of the expressions employed in the evaluation of the terms involved in the stiffness matrix $[K]$, and hence of the terms themselves, the resulting displacements exhibit small irregularities within localized areas that are apparently particularly resistant to elimination. It must be appreciated that the finite element method provides only an approximation to the actual behavior, involving, as it does, uniform stresses and strains within each triangle. Irregularities of some order must therefore be anticipated. The inadequacies of the method are particularly apparent at boundaries, the usual means employed to combat these being to provide many more, smaller, elements at, and in the vicinity of such boundaries. Such elaboration was considered unwarranted, and hopefully unnecessary, in the present study. A reduction of the displacement irregularities would probably be obtained by the use of greater numbers of elements, thereby proportionately reducing boundary effects; this possibility could not be investigated because of limitations on the time available for this study. Should such enlargement be contemplated then some procedure would be required to split the final matrix $[K]$ to more manageable halves.



(a) Configuration of general triangular element



(b) Typical upstream region of embankment-foundation



(c) Typical downstream region of embankment-foundation system

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FINITE ELEMENT GEOMETRY		PLATE NO. A1

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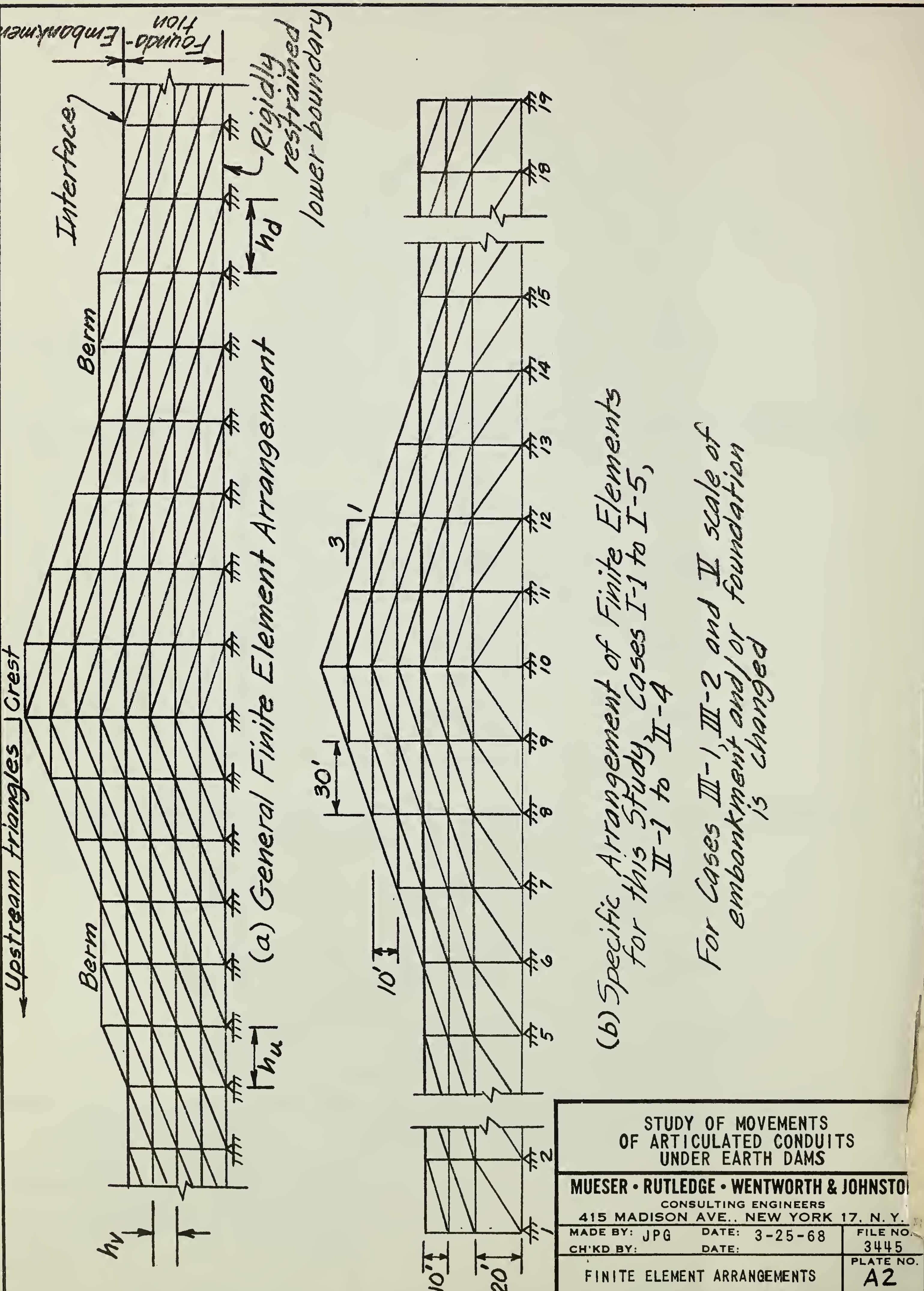

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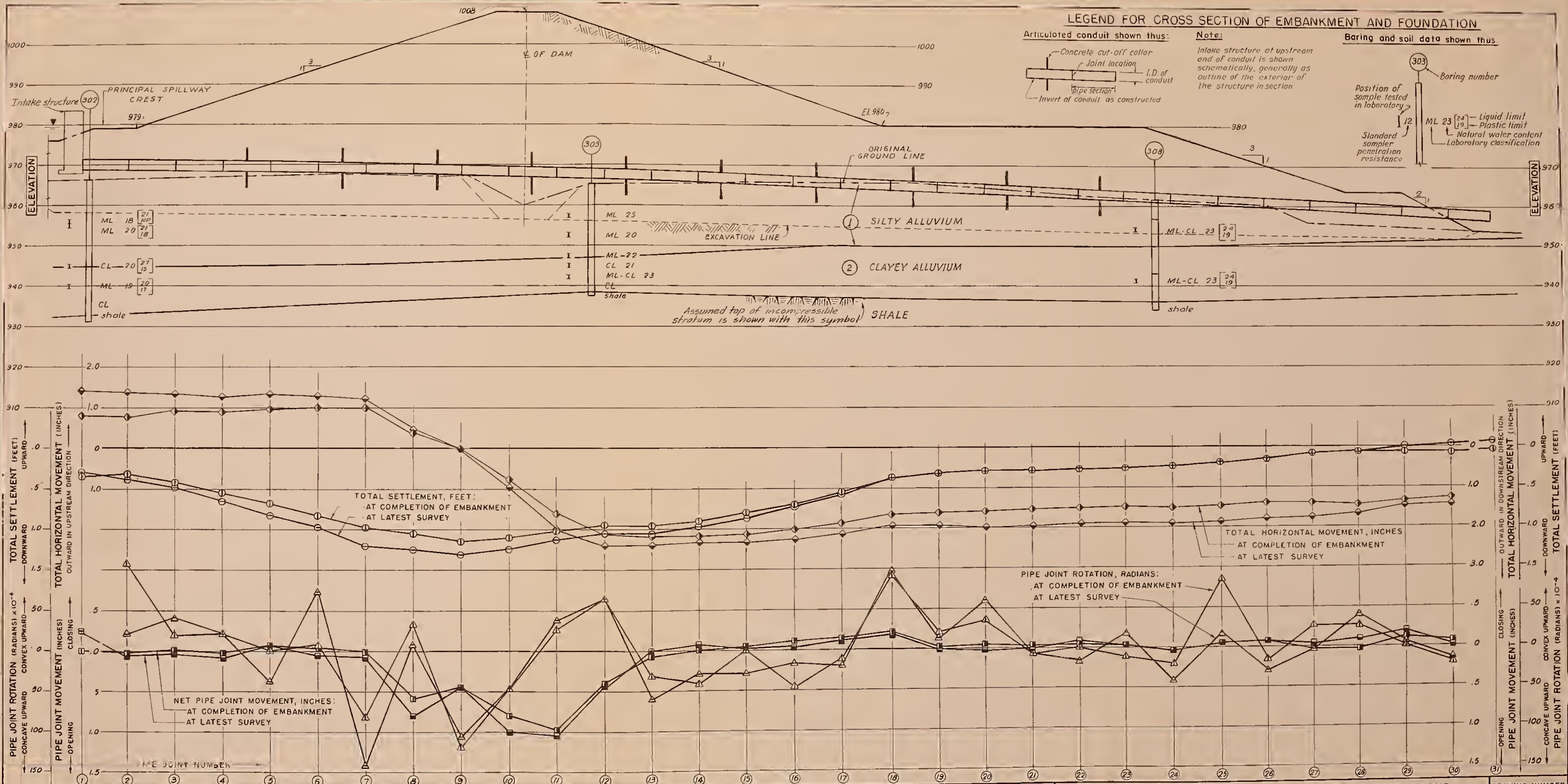

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DRAWING NUMBER 1

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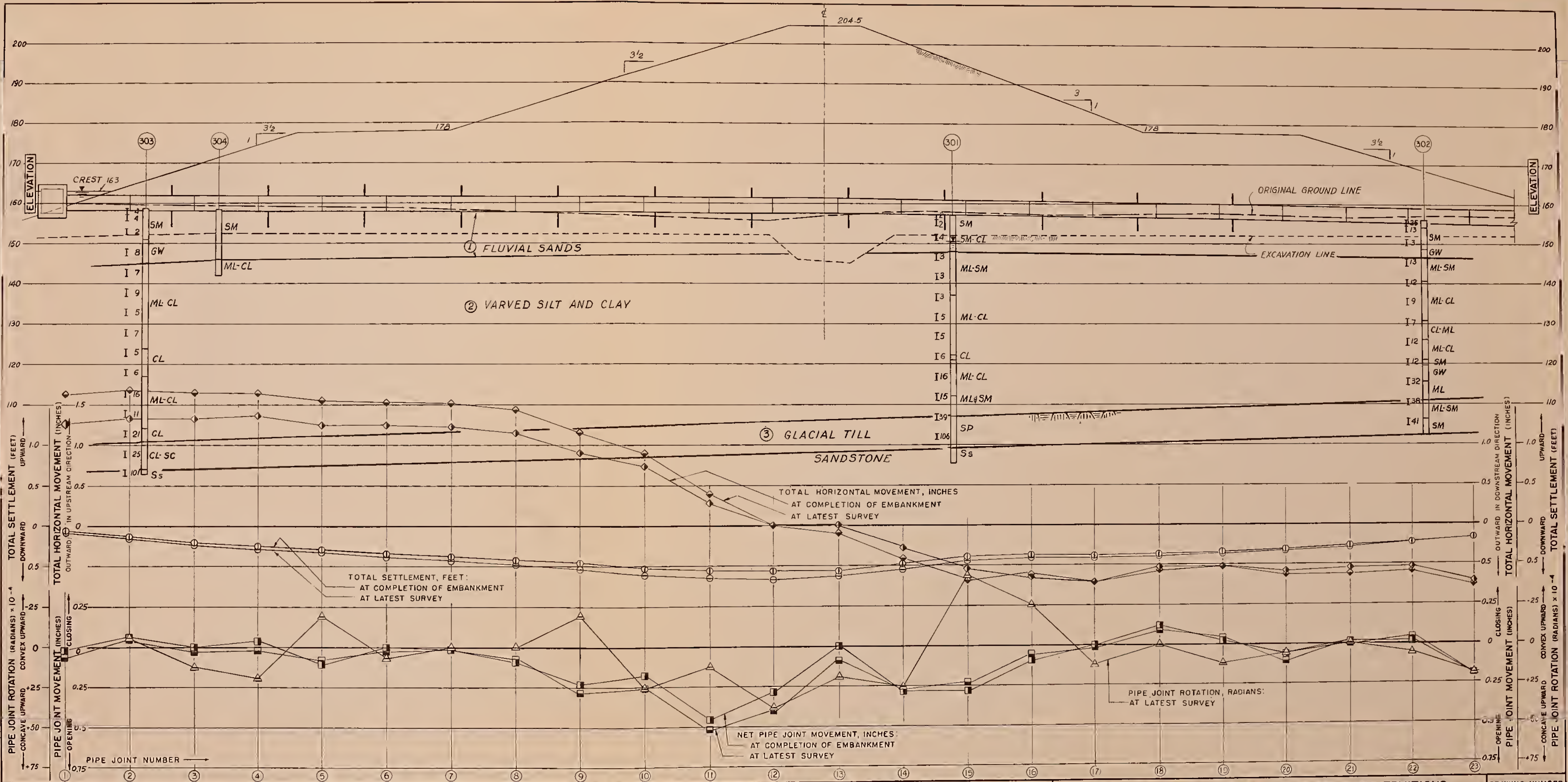

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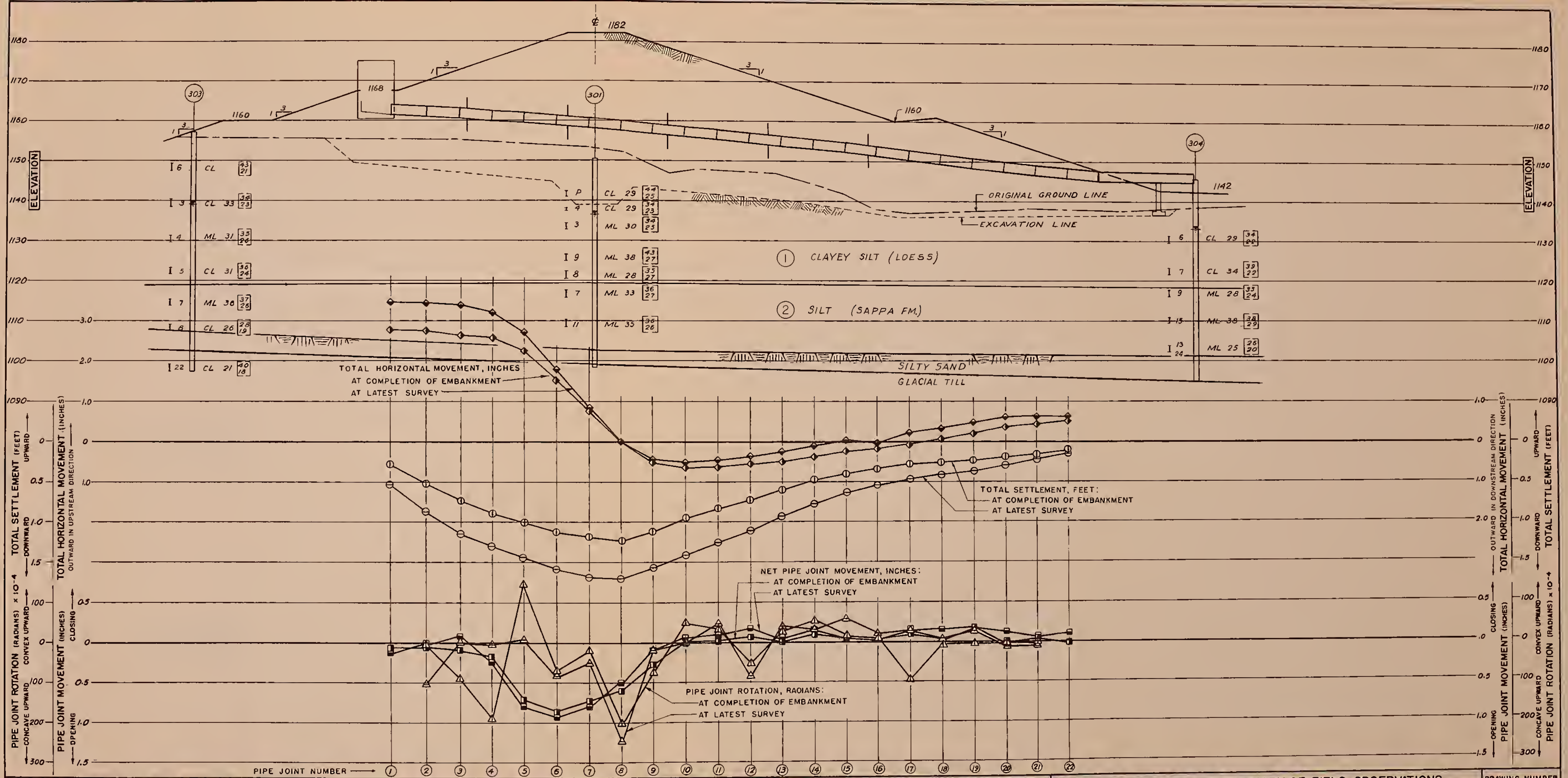

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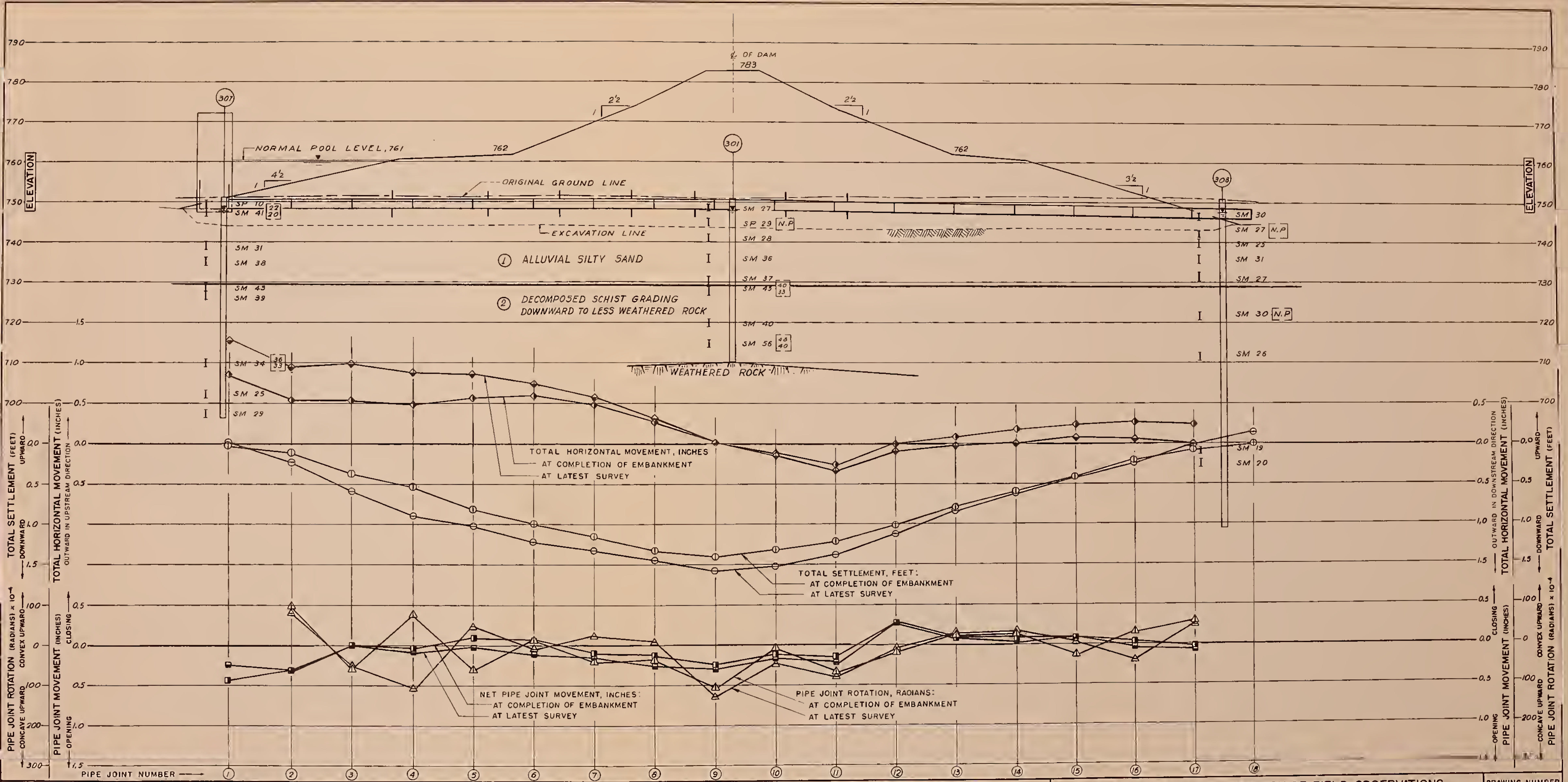

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STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS
DAM: HUFF CREEK No. 2A
LOCATION: GREENVILLE CO., SO. CAROLINA CASE NO: 4
SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'

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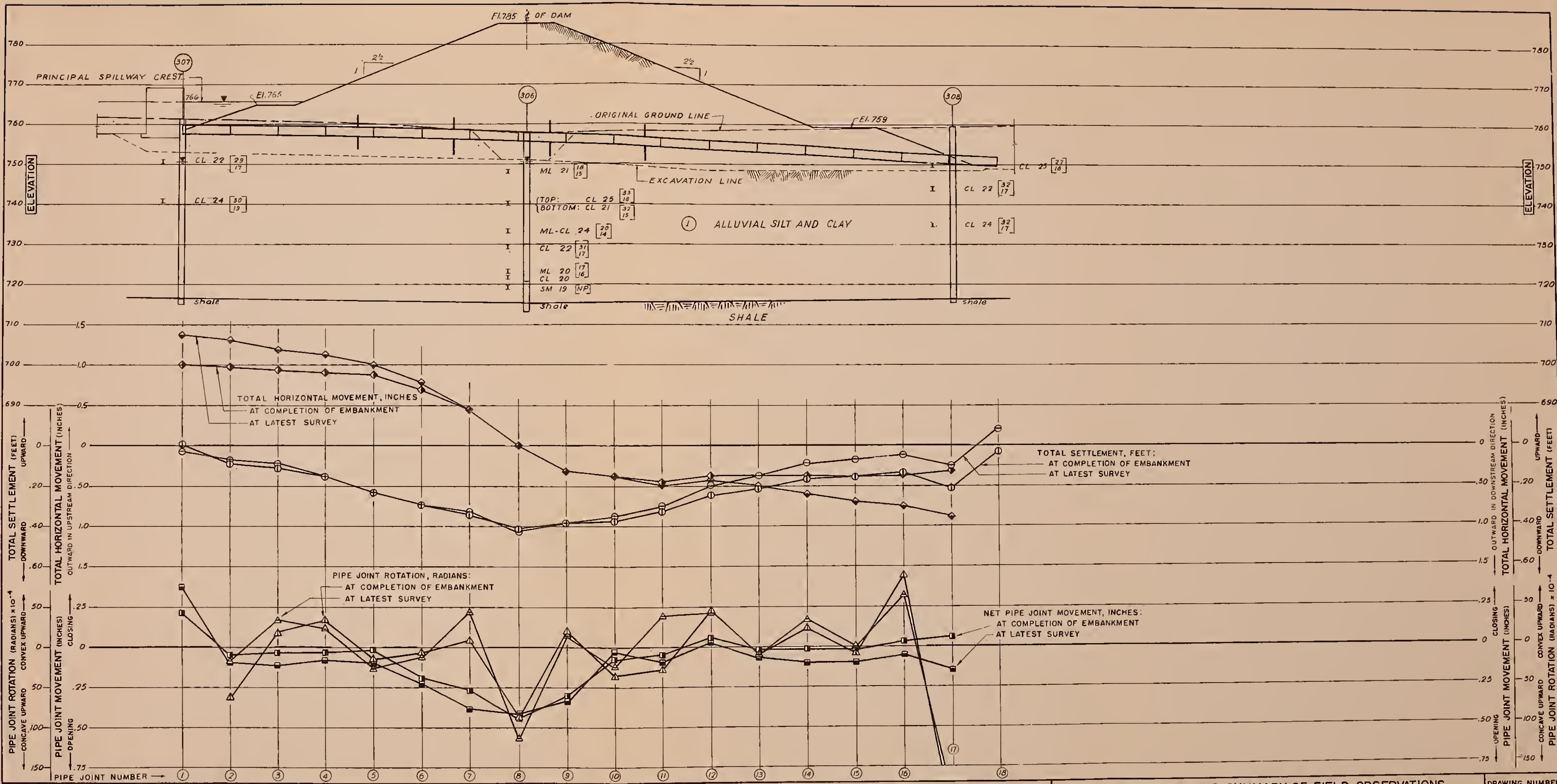
4



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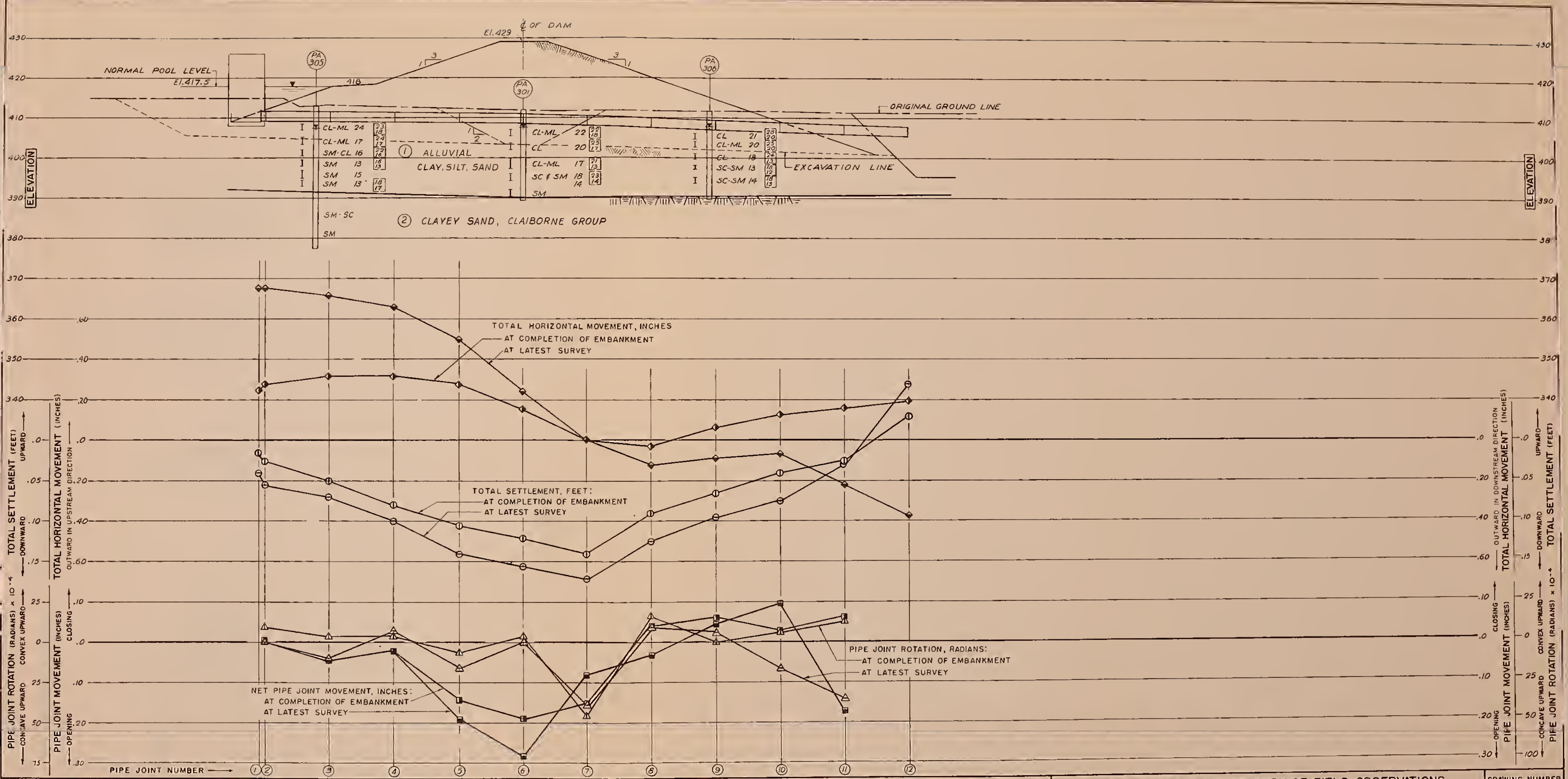
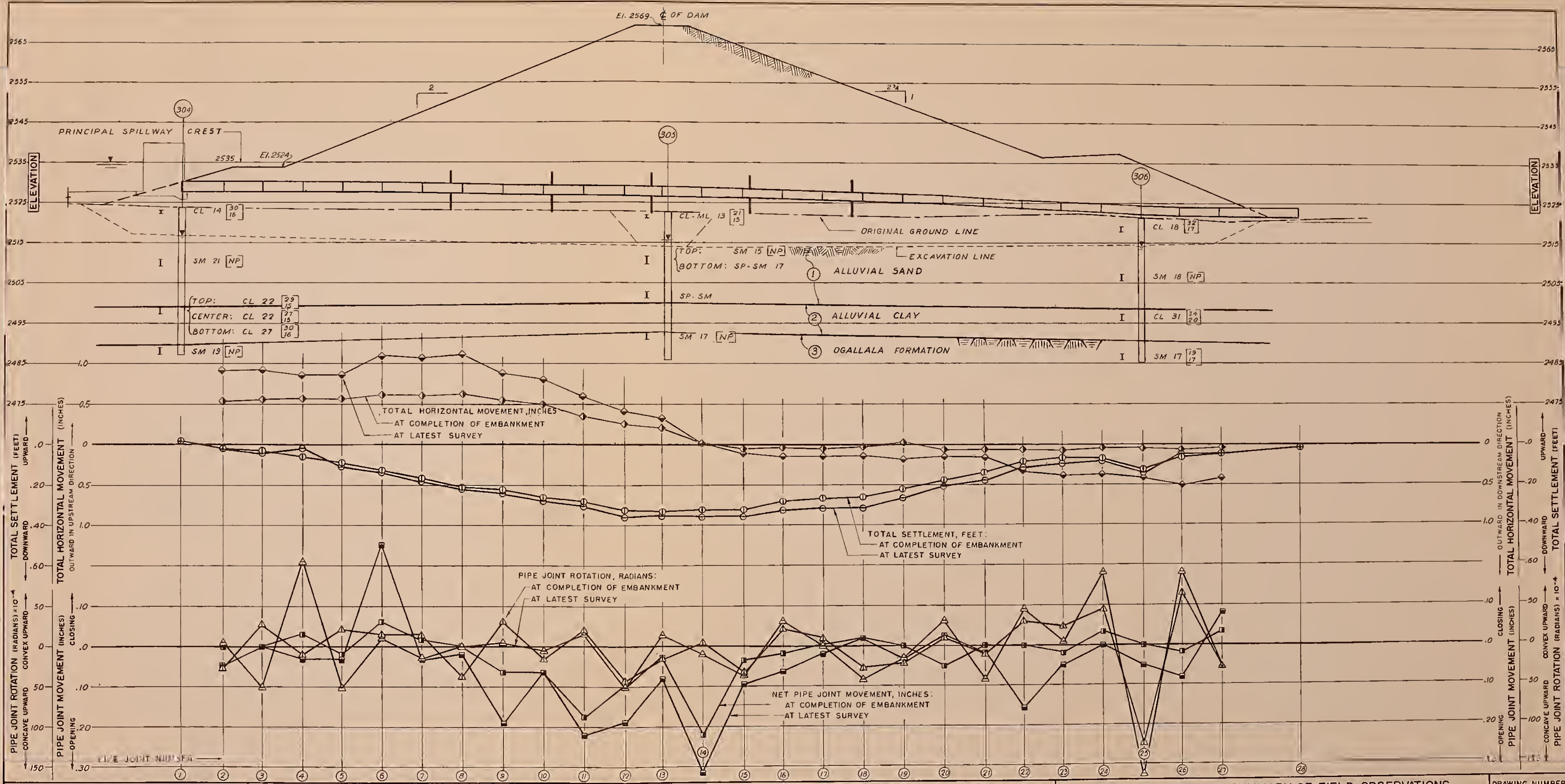
DRAWING NUMBER 5



EVALUATION BY PEP DATE August 1967 DRAWN BY HW & AR DATE 5-14-68 REVIEWED BY JPG DATE 5-17-68 MRWJ JOB NUMBER 3445	MUESER · RUTLEDGE · WENTWORTH & JOHNSTON CONSULTING ENGINEERS 415 MADISON AVENUE, NEW YORK, N.Y. 10017	STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS UNITED STATES DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS DAM: CYPRESS CREEK No. 8 LOCATION: WEAKLEY CO., TENNESSEE CASE NO: 6 SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'

DRAWING NUMBER 6



EVALUATION	BY PEP	DATE August 1967
DRAWN	BY HW & AR	DATE 5-14-68
REVIEWED	BY JPG	DATE 5-17-68

MRWJ JOB NUMBER 3445

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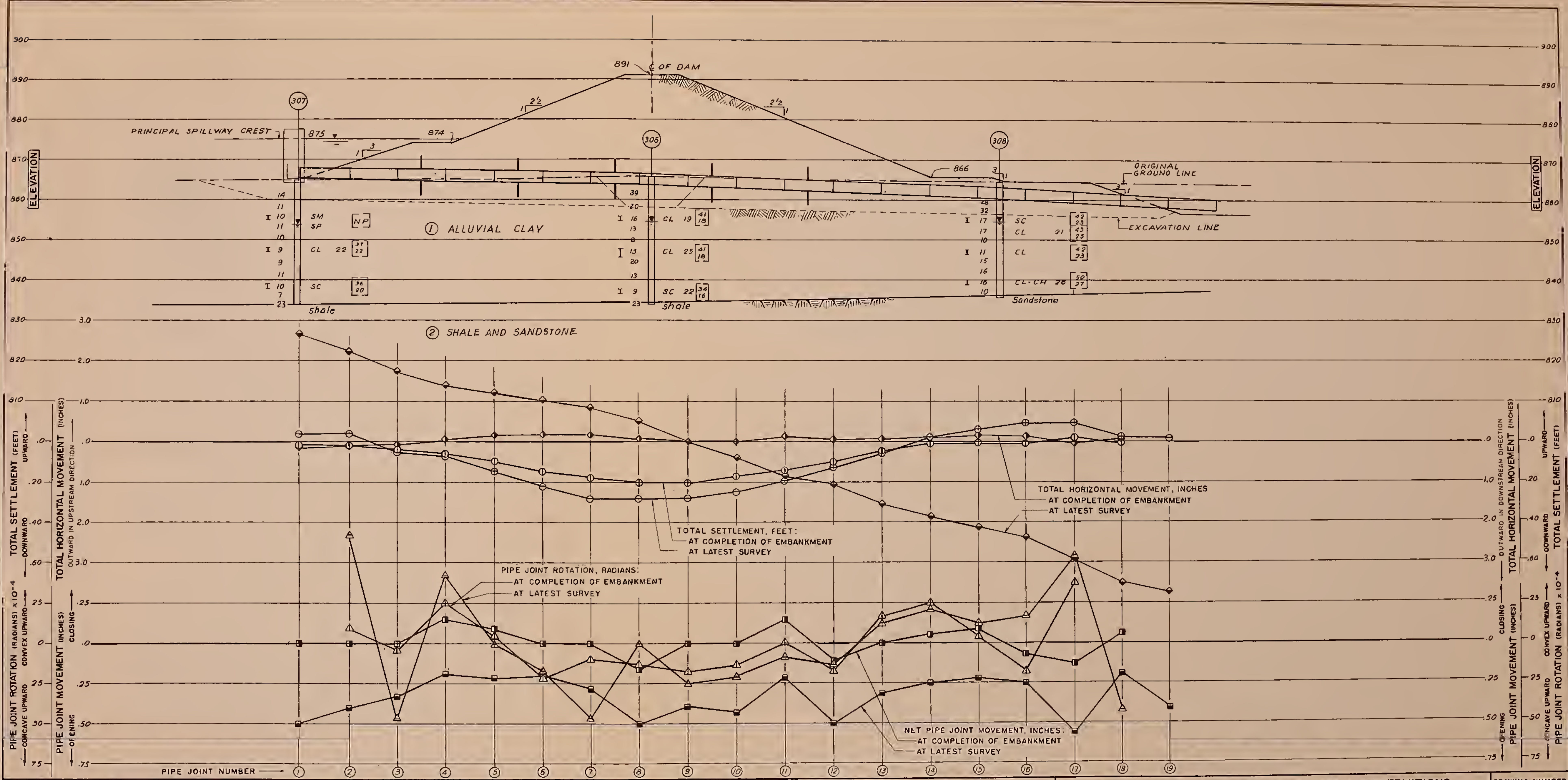
STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS
DAM: UPPER WASHITA RIVER No. 9
LOCATION: HEMPHILL CO., TEXAS
CASE NO: 7
SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'

DRAWING NUMBER
7



EVALUATION BY PEP	DATE August 1967	
DRAWN BY HW & AR	DATE 5-14-68	
REVIEWED BY JPG	DATE 5-17-68	
MRWJ JOB NUMBER 3445		

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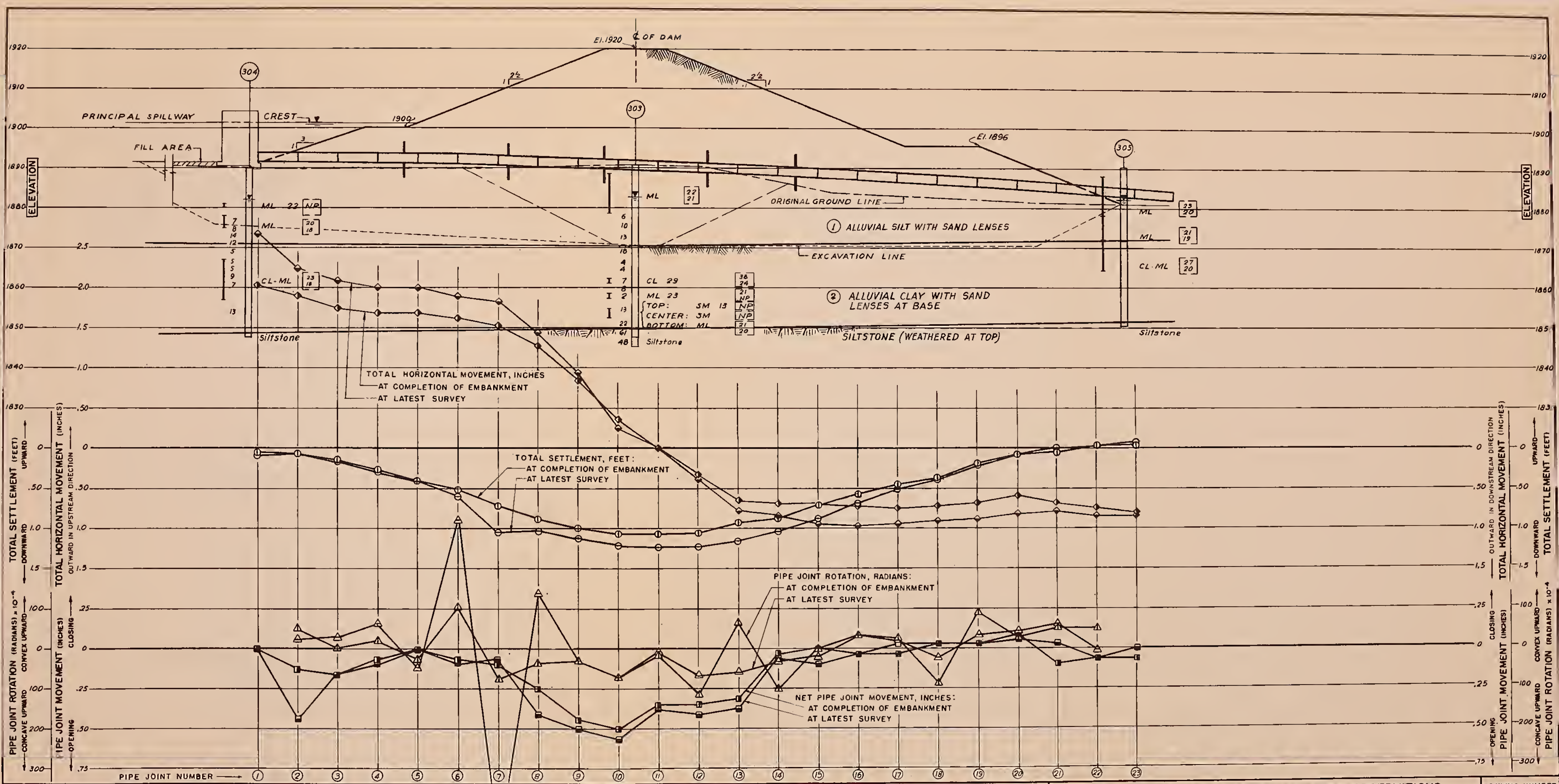
CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS
DAM: KICKAPOO-SANDY CREEK No. BJ-1
LOCATION: MURRAY CO., OKLAHOMA CASE NO: 8

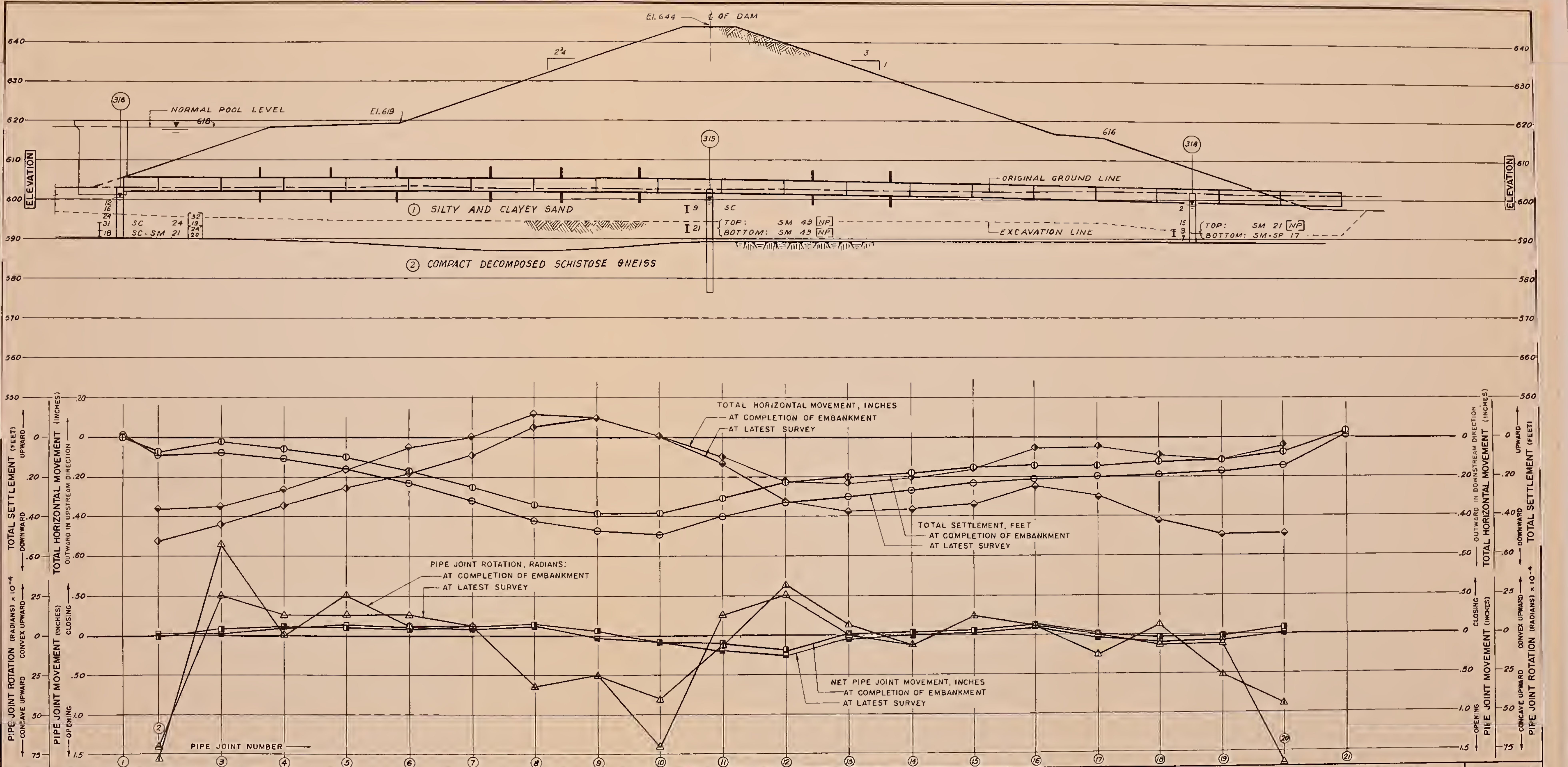
SCALE FOR CROSS SECTION: 0 10' 20' 30' 40'

DRAWING NUMBER 8

A



EVALUATION BY PEP DATE August 1967 DRAWN BY HW & AR DATE 5-15-68 REVIEWED BY JPG DATE 5-17-68 MRWJ JOB NUMBER 3445	MUESER · RUTLEDGE · WENTWORTH & JOHNSTON CONSULTING ENGINEERS 415 MADISON AVENUE, NEW YORK, N.Y. 10017	STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS UNITED STATES DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS DAM: QUARTERMASTER CREEK No. 1 LOCATION: ROGER MILLS CO., OKLAHOMA CASE NO: 9 SCALE FOR CROSS SECTION:
DRAWING NUMBER 9			



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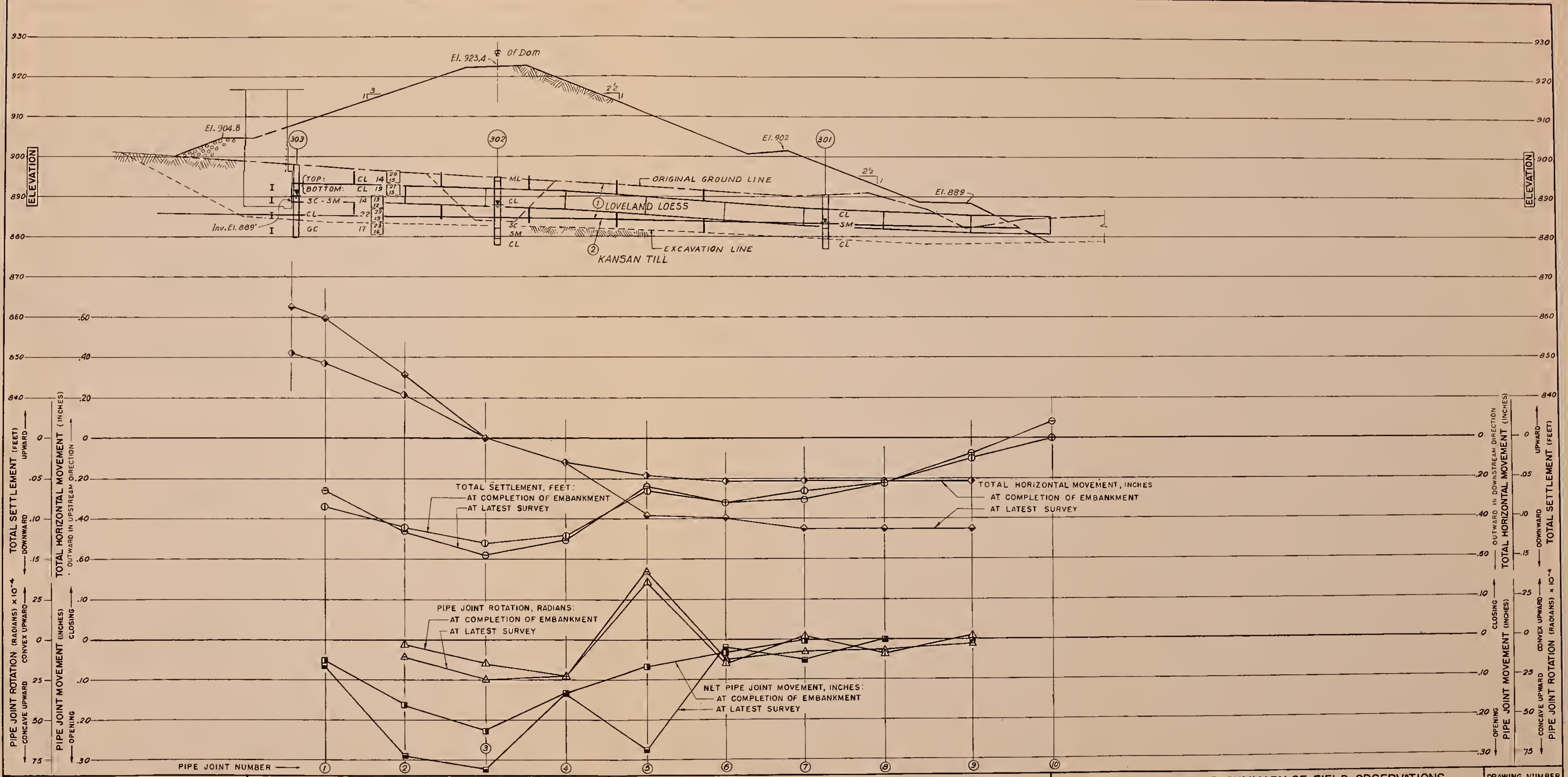
DRAWING NUMBER 10



EVALUATION BY PEP	DATE August 1967	
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REVIEWED BY JPG	DATE 5-17-68	

MRWJ JOB NUMBER 3445

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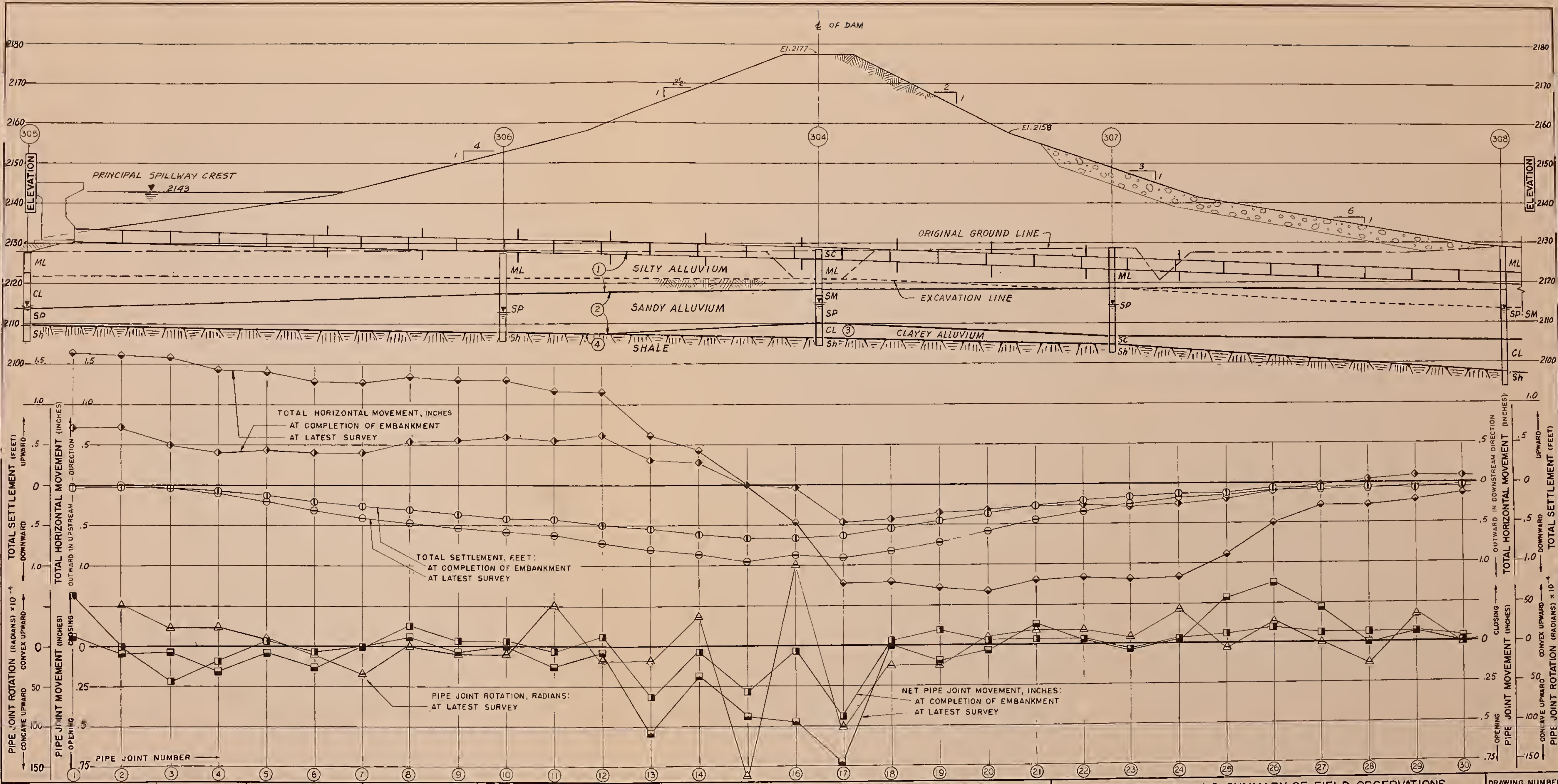
SOIL CONSERVATION SERVICE

CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS
DAM: WHITE CLAY-BREWERY-WHISKEY CREEK No. 6
LOCATION: ATCHISON CO., KANSAS
CASE NO: 11

SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'

DRAWING NUMBER
11



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REVIEWED BY JPG	DATE 5-17-68	
MRWJ JOB NUMBER 3445		

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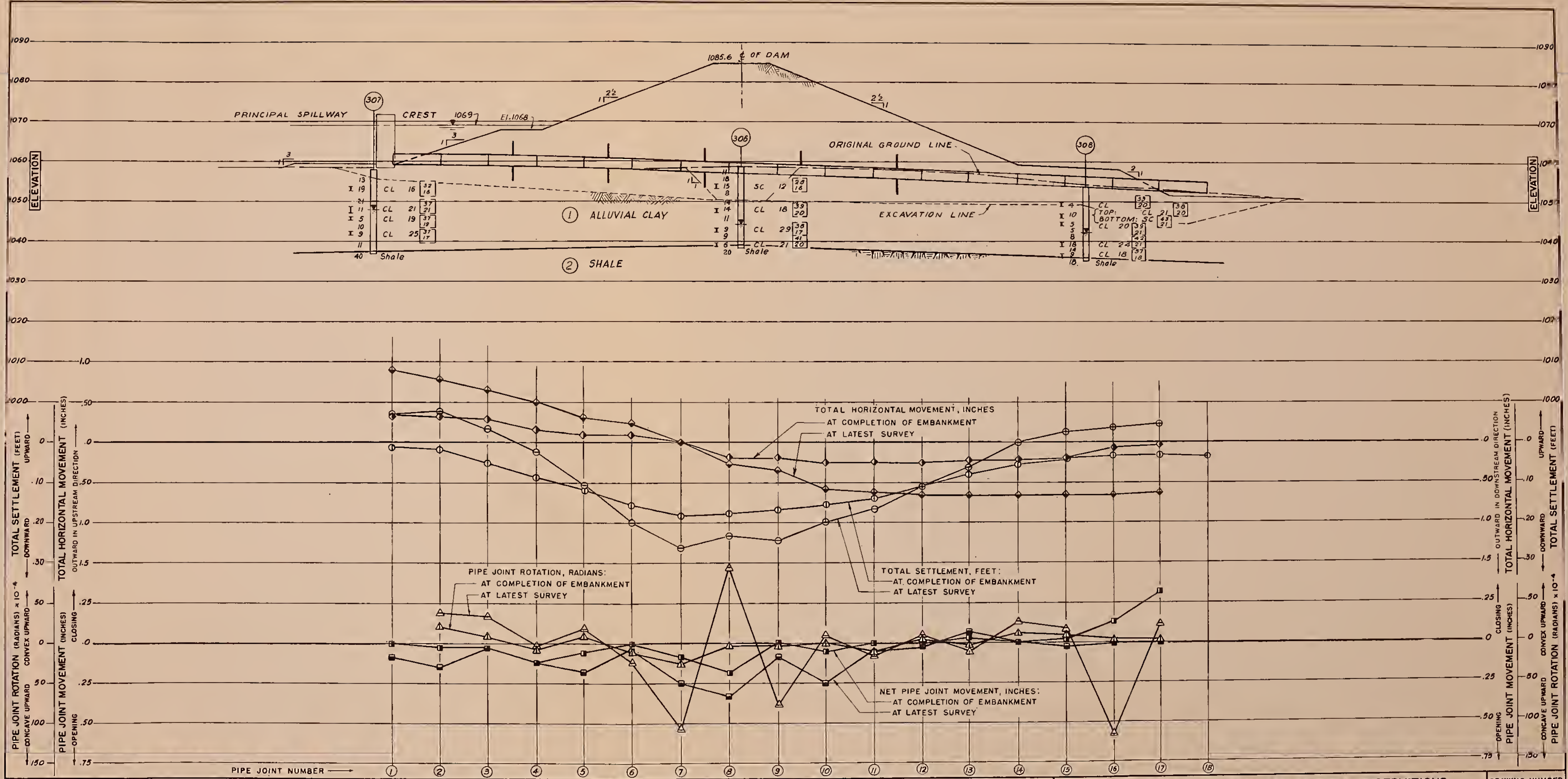
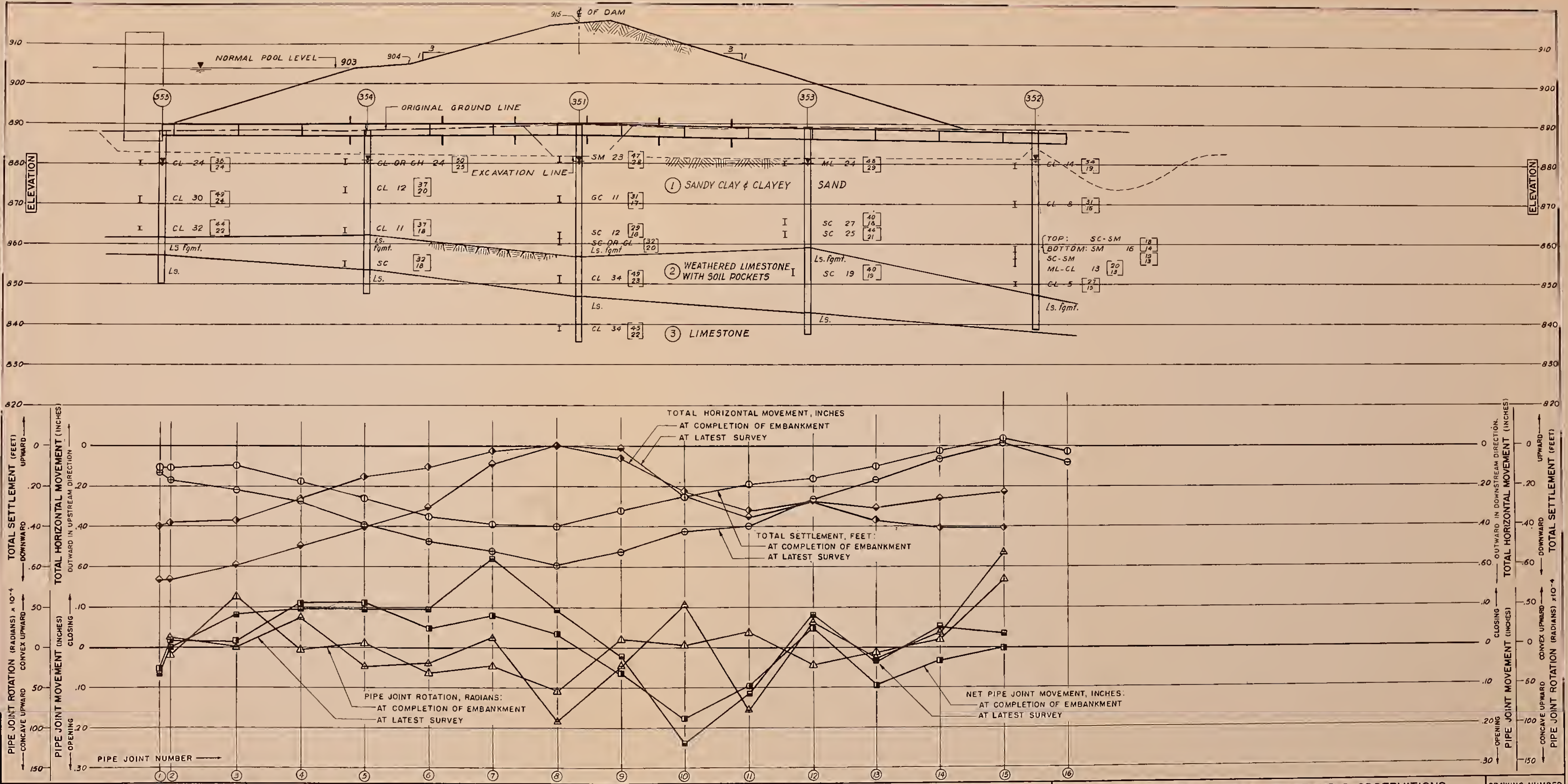
CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS
DAM: UPPER WASHITA RIVER No. 57
LOCATION: ROGER MILLS CO., OKLAHOMA CASE NO: 12
SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'

DRAWING NUMBER
12

A



EVALUATION BY PEP DATE August 1967 DRAWN BY HW & AR DATE 5-16-68 REVIEWED BY JPG DATE 5-17-68 MRWJ JOB NUMBER 3445	MUESER · RUTLEDGE · WENTWORTH & JOHNSTON CONSULTING ENGINEERS 415 MADISON AVENUE, NEW YORK, N.Y. 10017	STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS UNITED STATES DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS DAM: UPPER RED ROCK CREEK No. 33 LOCATION: GARFIELD CO., OKLAHOMA CASE NO: 13 SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'	DRAWING NUMBER 13



EVALUATION BY PEP	DATE August 1967	
DRAWN BY HW & AR	DATE 5-15-68	
REVIEWED BY JPG	DATE 5-17-68	

MRWJ JOB NUMBER 3445

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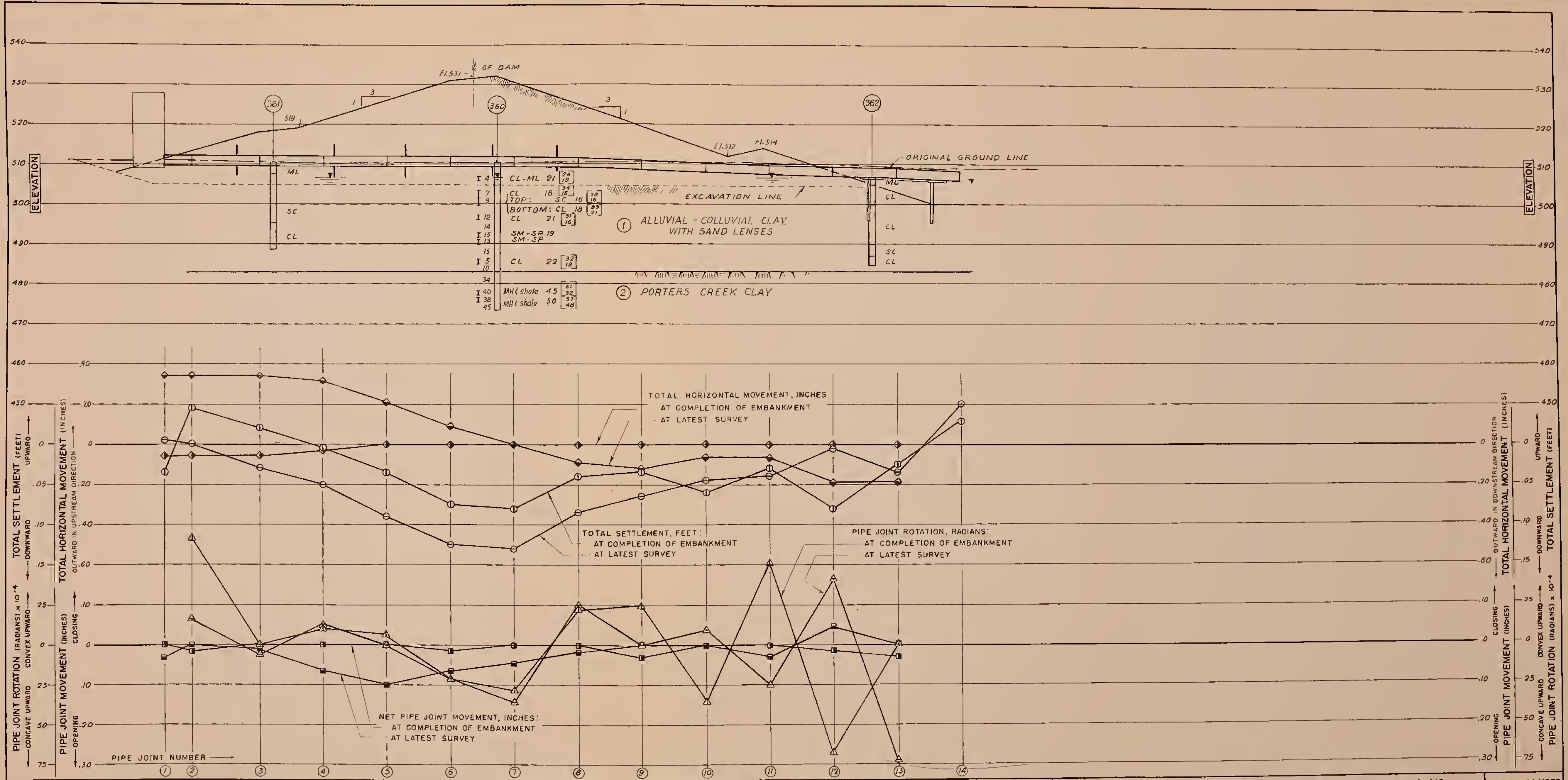
SOIL CONSERVATION SERVICE

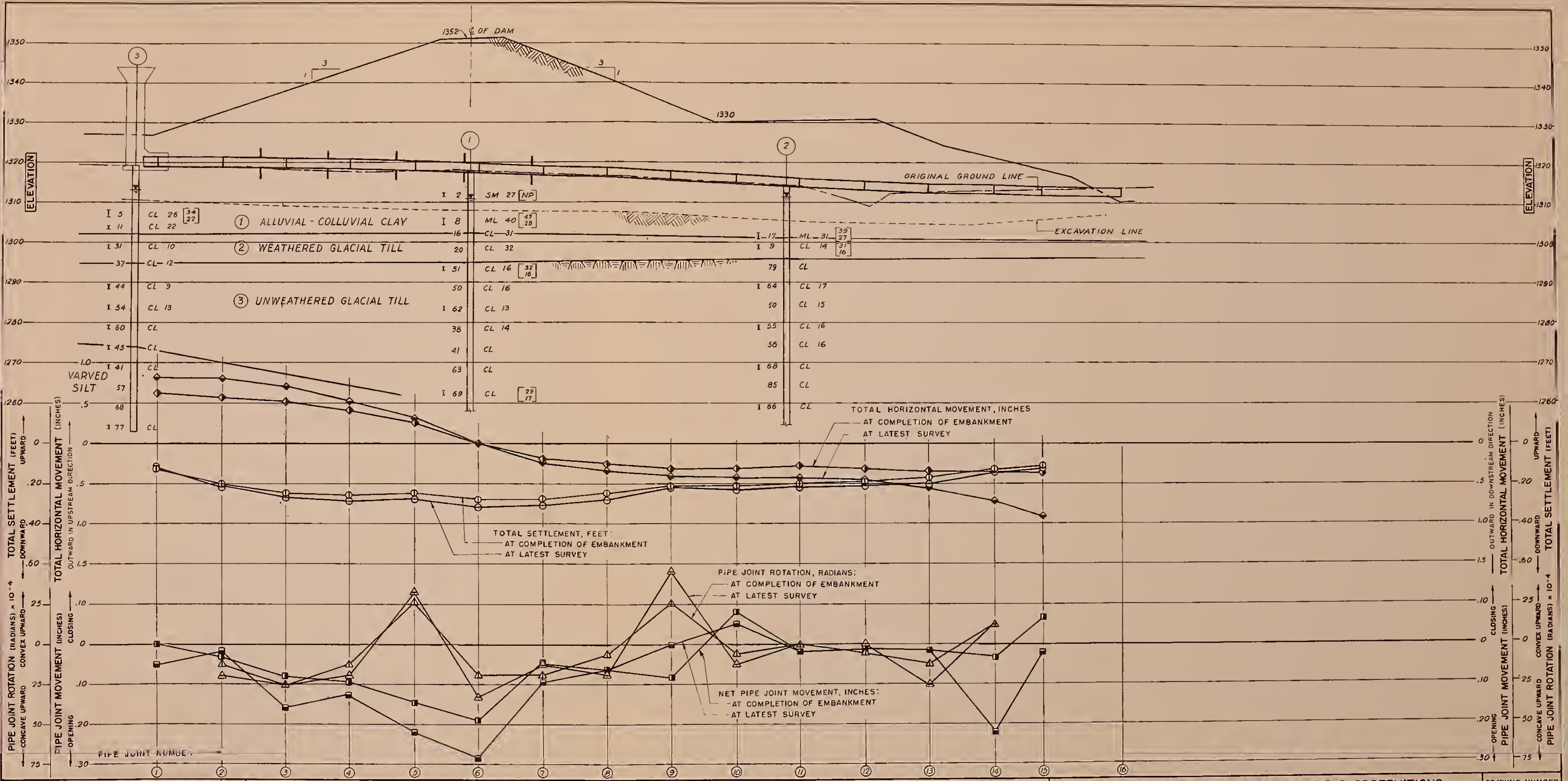
CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS
DAM: BRISTOW'S CREEK No. 1
LOCATION: ETOWAH CO., ALABAMA CASE NO: 14
SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'

DRAWING NUMBER
14



EVALUATION BY PEP DATE August 1967 DRAWN BY HW B AR DATE 5-16-68 REVIEWED BY JPG DATE 5-17-68 MRWJ JOB NUMBER 3445	MUESER · RUTLEDGE · WENTWORTH & JOHNSTON CONSULTING ENGINEERS 415 MADISON AVENUE, NEW YORK, N.Y. 10017	STUDY OF MOVEMENTS OF ARTICULATED CONDUITS UNDER EARTH DAMS ON COMPRESSIBLE FOUNDATIONS UNITED STATES DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS DAM: PORTER'S CREEK No. II LOCATION: TIPPAH CO., MISSISSIPPI CASE NO: 15 SCALE FOR CROSS SECTION: 0 10' 20' 30' 40'

DRAWING NUMBER 15



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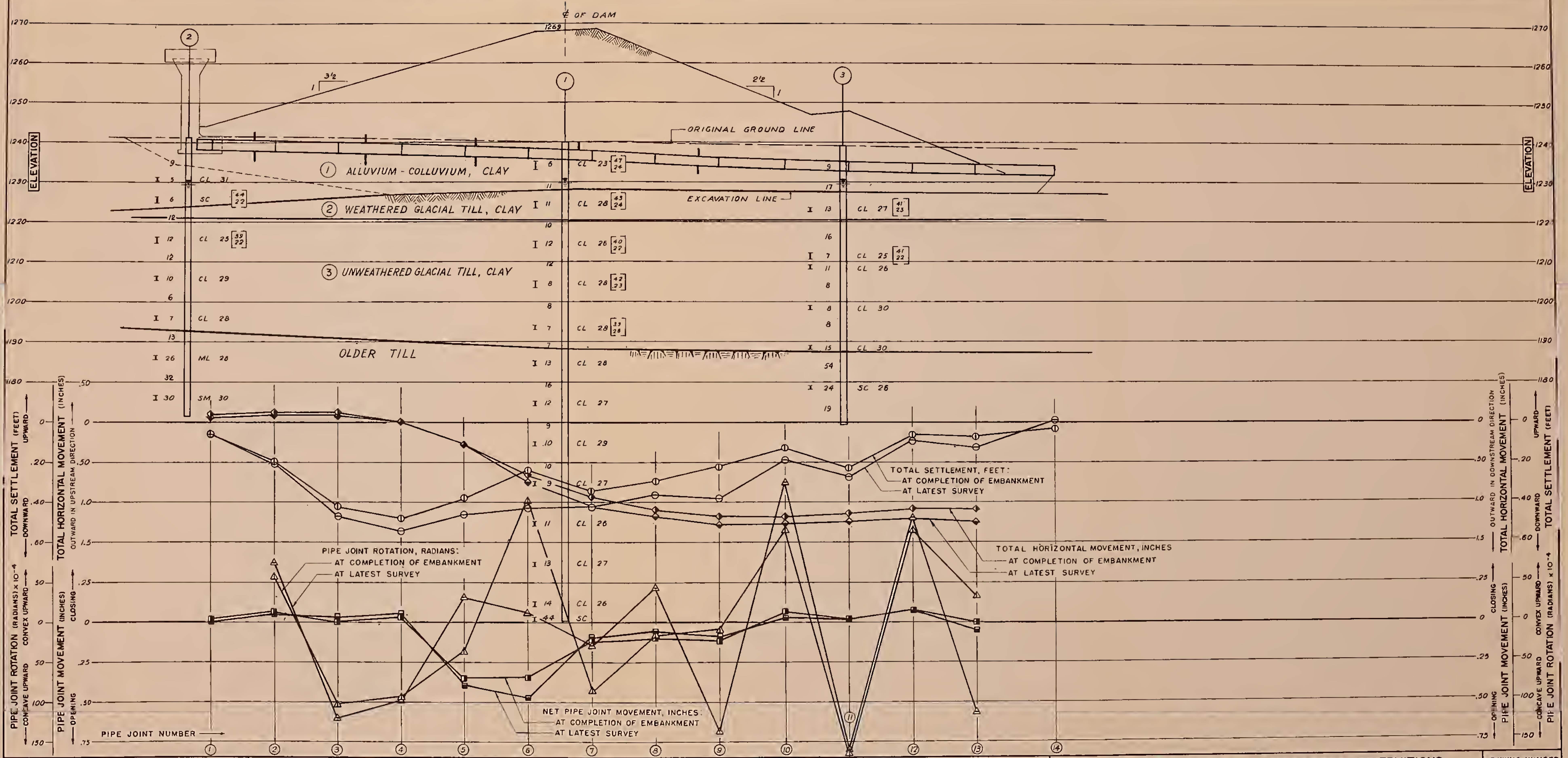
SOIL CONSERVATION SERVICE

CROSS SECTION AND SUMMARY OF FIELD OBSERVATIONS
DAM: TEWAUKON No. TI-A
LOCATION: SARGENT CO., N. DAKOTA   CASE NO: 16
SCALE FOR CROSS SECTION: 0 5' 10' 20' 30' 40'

DRAWING NUMBER 16



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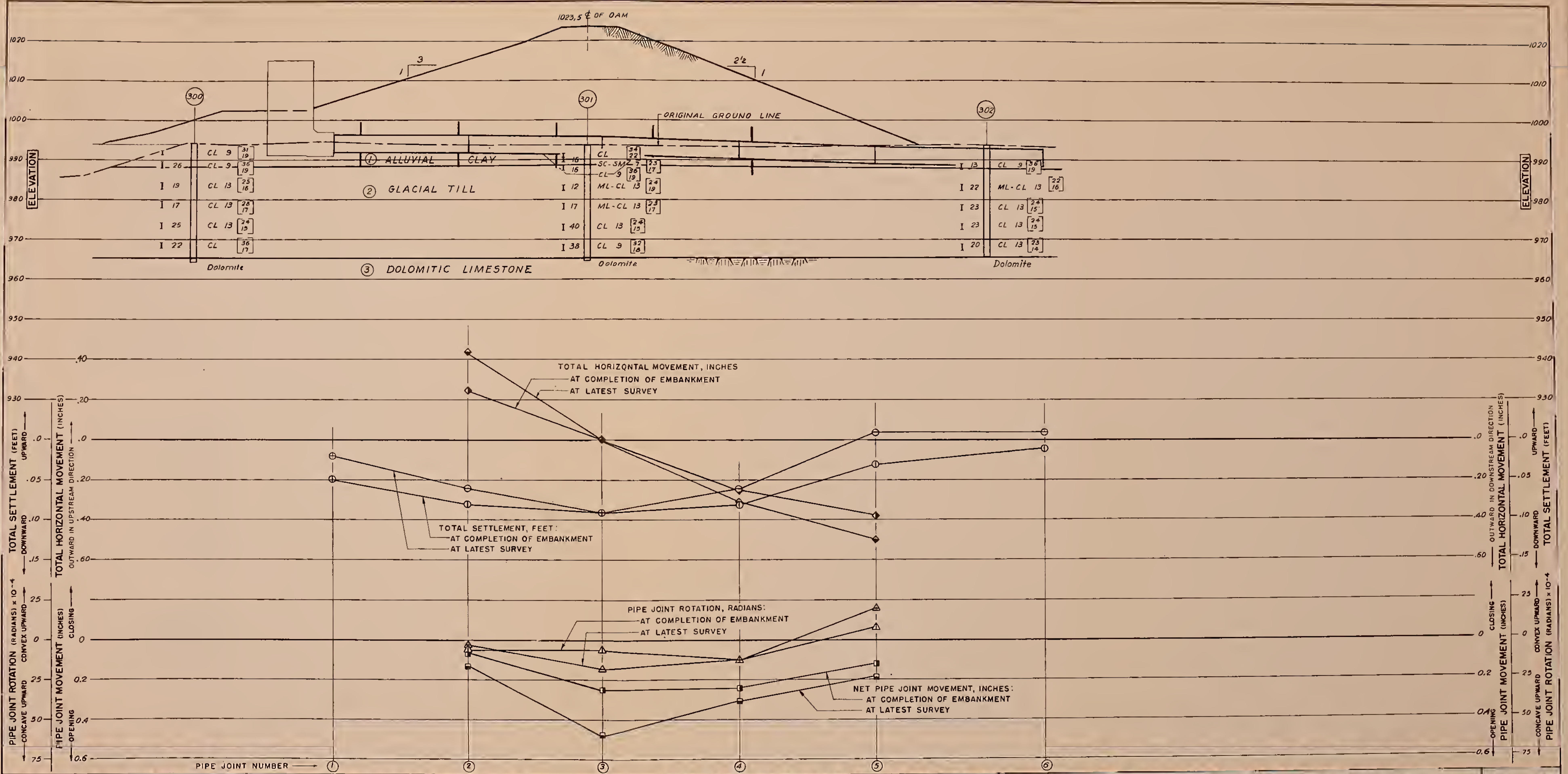
DRAWING NUMBER 17



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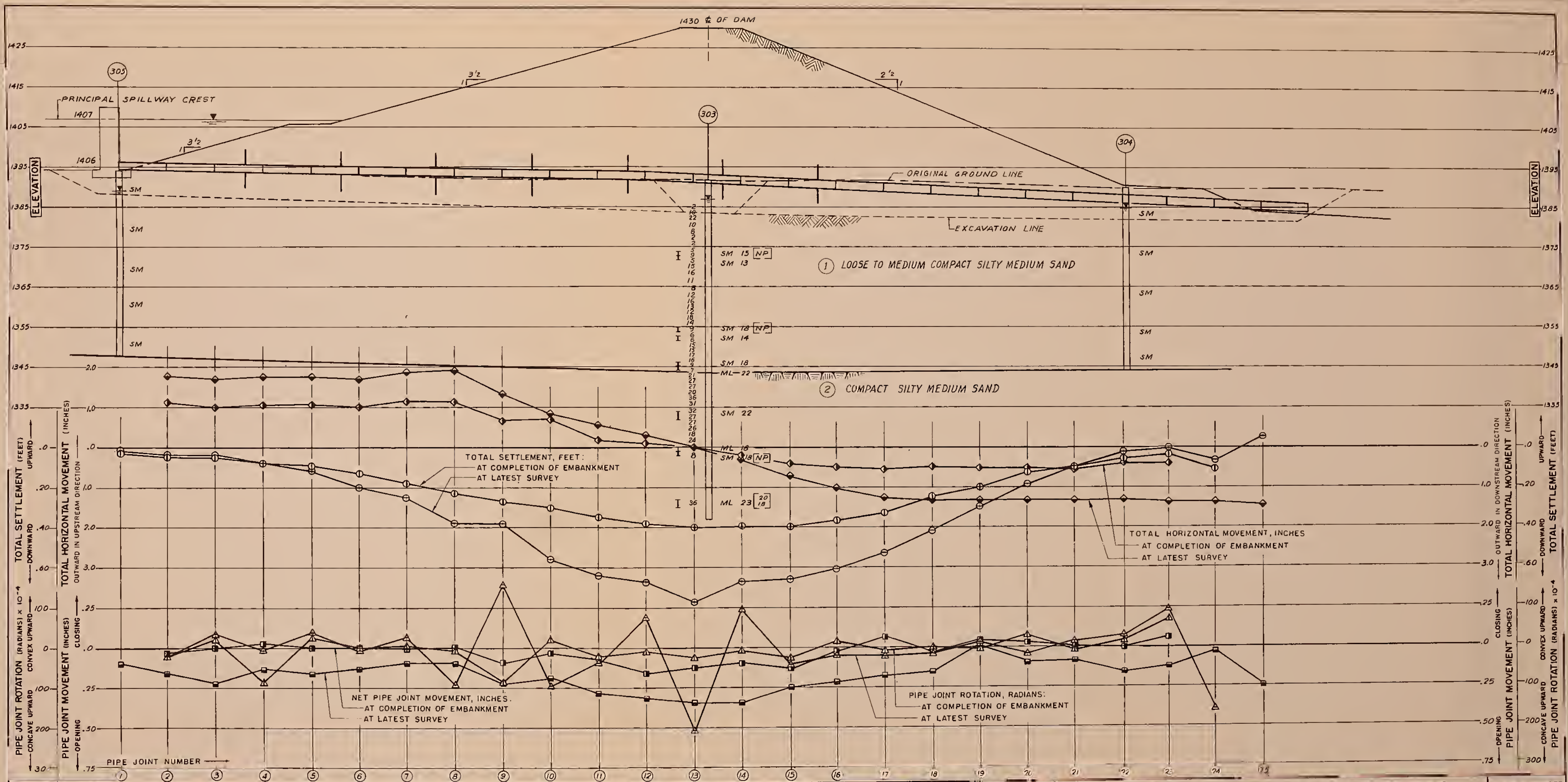
DRAWING NUMBER 18



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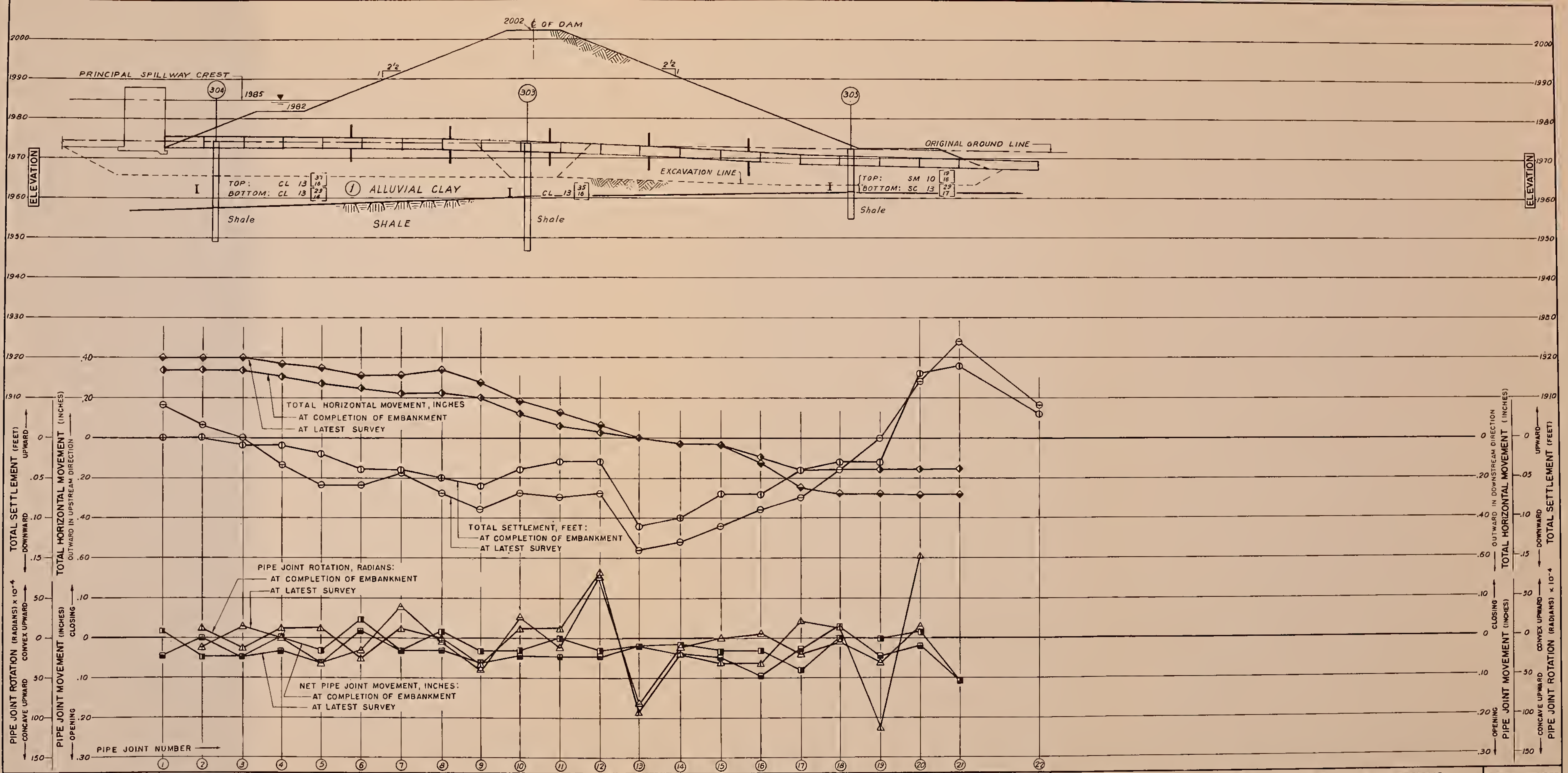
DRAWING NUMBER 19



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